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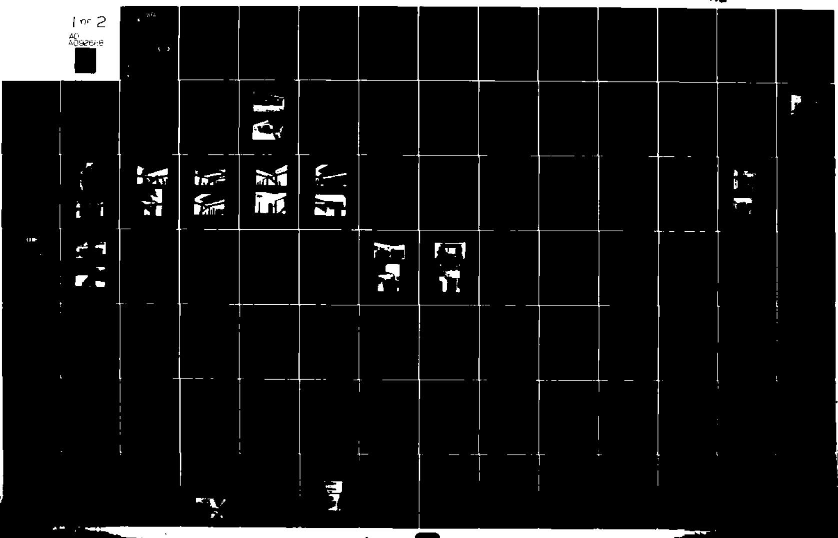

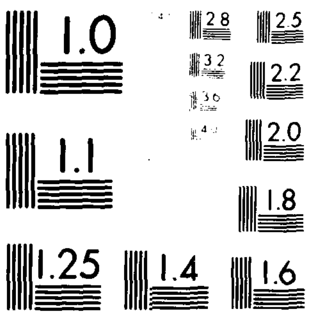

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FINAL REPORT

FEMA Contract DCPA01-78-C-0223

June 1980

Approved for Public Release; Distribution Unlimited

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FINAL REPORT

By

A. Longinow
R. P. Joyce

for

Federal Emergency Management Agency
Washington, D.C. 20472

June 1980

Approved for Public Release; Distribution Unlimited.

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The results of an experimental study which was conducted to determine the strength of residential, wood floor systems over basements to uniform static load are presented in this report. A single family residence which was slated for demolition was acquired and load tested. Two load tests were conducted. The first load test was concerned with the strength of the as-built floor system. One half of the floor system was instrumented and loaded to collapse. Failure was experienced as 185.4 psf.

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The second load test was concerned with the strength of an expediently upgraded floor system. The remaining half of the floor system was upgraded by placing a studwall in the longitudinal direction halfway between the exterior wall and the girder in each of the two spans. The floor was loaded to 559.3 psf. At this load the test was terminated due to reasons of safety. The floor system did not fail. Additional tests were conducted in the laboratory on the unfailed portions of the floor system. This consisted of three "simple beam" tests of samples consisting of two joists with flooring attached. The load was uniformly distributed. The loading in each case was accomplished using solid concrete block.

This report includes experimental results, analysis of experimental results and predicted collapse loads using a simplified prediction method.

Probability of people survival estimates are included for two shelter conditions. In the first, the shelter is assumed to consist of the as-built basement with one foot of soil over the floor for radiation protection. In the second, the shelter is assumed to consist of the expediently upgraded basement with one foot of soil for radiation protection. The probability of people survival is estimated against blast effects produced by the detonation of a single 1-MT nuclear weapon.

FOREWORD

This is the final report on IIT Research Institute project J6451 entitled "Load Tests of a Wood Floor Over a Basement". This study was performed for the Federal Emergency Management Agency (FEMA) under Contract DCPA01-78-C-0223. The study was initiated on June 13, 1978 and completed June 6, 1980. The work was performed by A. Longinow, R. Joyce, D. Hrdina and C. Foxx of the Engineering Division. The work was monitored by D. A. Bettge of FEMA.

The test building used in this study was provided by the U.S. Department of the Interior, National Park Service, Indiana Dunes National Lakeshore. The cooperation of Mr. J. R. Whitehouse and Mr. H. Culp of the Indiana Dunes National Lakeshore in making the building available, and taken an active interest during the conduct of tests is gratefully acknowledged.

Respectfully submitted,
IIT RESEARCH INSTITUTE

A. Longinow
Project Engineer

Approved by:

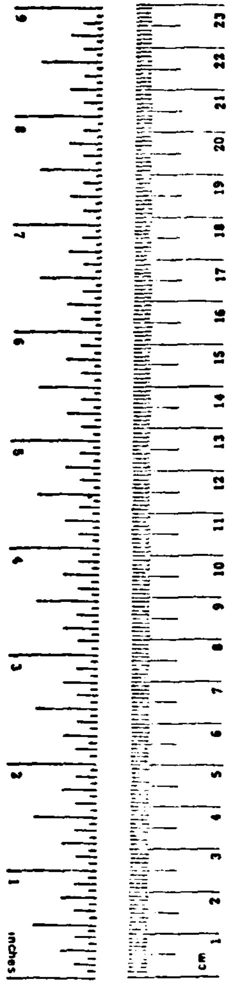
Renny S. Norman
Manager
Engineering Mechanics

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

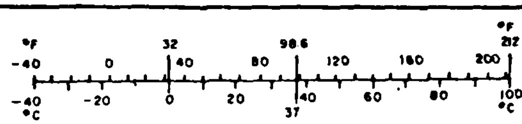
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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1. INTRODUCTION, SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

1.1 Introduction

The current posture for national survival in the event of a nuclear war includes Crisis Relocation Planning (CRP). This involves (a) evacuation of the major portion of the population from high risk to low risk areas, and (b) providing nuclear weapons effects protection for the contingent of key workers, who would remain behind to staff essential industries.

CRP assumes that a crisis period will precede any future conflict. During this period evacuation and relocation would take place. Also, low level weapon effects shelters would be prepared by upgrading existing facilities in host areas. High level weapon effects shelters would be prepared by upgrading basements of existing facilities, or building special shelters, in high risk areas.

The above-mentioned "upgrading" refers to expedient measures that may be quickly employed to strengthen existing facilities and thus to gain protection against nuclear weapon effects. In low risk (host) areas, nuclear weapon effects would mostly consist of prompt effects with overpressures in the range of about 2 psi and less plus the possible delayed effect of fallout and fires. It is believed that most existing structures, and especially basements are capable of being upgraded to provide the needed protection in host areas.

The objective of this study was to load test the floor over the basement of a framed single-family dwelling and determine its static load capacity in the as-built and upgraded conditions.

1.2 Summary

A single-family dwelling with a full basement that was slated for demolition was acquired for test purposes. Two load tests were conducted. The first test was concerned with the strength of the as-built floor system. One-half of the floor system in the longitudinal direction was instrumented and loaded to collapse. Instrumentation consisted of (a) deflection measurements approximately at midspan under each of the two joist spans, and (b) deflection measurements of the girder. The floor was loaded using solid concrete block. Failure was experienced at a uniform load of 185.4 psf and consisted of ruptures of five joints. With one possible exception, the ruptures were initiated at a flaw such as a knot, split or saw cut.

In the second load test, the remaining half of the floor system was expediently upgraded by placing a studwall in the longitudinal direction halfway between the exterior wall and the girder in each of the two spans. The floor was instrumented to yield joist and girder deflections. It was loaded in a systematic manner to 559.3 psf. At this load the test was terminated due to reasons of safety. The floor system did not fail and an examination of the structure after the load was removed revealed no visible signs of distress.

Additional tests were conducted in the laboratory on the unfailed portions of the floor system. This consisted of three "simple beam" tests of samples consisting of two joists with flooring attached. The load was uniformly distributed. The loading in each case was accomplished using solid concrete block.

This report includes experimental results, analysis of experimental results and predicted collapse loads using a simplified prediction method. Probability of people survival estimates against the effects of blast produced by the detonation of a 1-MT weapon are included for two shelter conditions. In the first, the shelter is assumed to consist of the as-built basement. In the second it is assumed to consist of the expediently upgraded basement. Soil cover with a depth of 1 foot for radiation protection is considered as one of the options.

1.3 Conclusions

Construction of existing frame single-family residences with basements differs widely and therefore firm conclusions on the behavior of the class of such structures cannot be derived from a single house. The following are observations as they relate to this particular house.

1. This floor system was not typical by current design practice. The joists were smaller and the spacing was wider than currently recommended. The floor was also unusual in that the subfloor and the finish floor consisted of tongue and groove boards running in the same direction.

2. This floor system was stronger than could be judged by the design load (40 psf) prevalent in this area (Tremont, Indiana). The failure load was 185.4 psf. The safety factor was thus 185.4/40 = 4.64.

3. During the first load test the upper story walls contributed somewhat in carrying part of the load applied to the floor, see joist 14W in Figure 19. This however is not expected to be typical since the upper story was unusual in that the interior walls consisted of 2 x 6 inch joists on 24 inch centers with 1 inch tongue and groove boards on both sides.

4. Experimental results indicate that some load sharing between the joists existed by virtue of the flooring. The floor therefore exhibited some two-way action, see Figures 18, 19, 29, and 30. The extent of this load sharing could not be determined.

5. Laboratory tests on the "simple beam" floor samples conducted in this study indicate that the major strength was in the joists. Composite action between the joists and the flooring was negligible (see Appendix C). Little or no composite action was observed in tests conducted at Waterways Experiment Station (WES) (Ref 1) and at Scientific Service Incorporated (SSI) (Ref 2). However partial composite action of a subfloor and finish floor with the supporting joists has been observed in tests of floor systems common to residential construction (Ref 3).

6. In future load tests of full-scale structures using concrete block, block should be carefully spaced to avoid or minimize contact between individual blocks when the structure deflects. Otherwise the load will not be uniformly distributed. For tests requiring "large" loads, such as the second load test conducted in this study, the use of concrete block should be avoided since it is difficult to avoid contact between blocks.

7. The studwall upgrading concept used in this study appears to be effective in increasing the strength of the floor system.

8. Additional upgrading would be required under actual shelter conditions. This would include blocking of windows, mounding the peripheral walls which protrude about 1 foot above grade, and bracing the walls internally to avoid cracking and possible dislocation of the unreinforced block walls.

9. Wood-frame structures are designed on the basis of allowable stresses which are set low enough that about 95 percent of the members found in a given lumber grade are able to carry the design load safely. For this reason, basements of framed residential dwellings have reserve strength and offer a shelter potential for host areas against low level blast effects.

1.4 Recommendations

1. Since basements of existing framed dwellings differ widely there is a need to conduct additional tests on a representative sample of such structures for as-built and upgraded conditions. Such tests should strive to identify modes of failure, particular weaknesses, and debris production expecially under dynamic loading conditions.

2. The subject of interactions between individual members of the floor assembly needs further study. We need to know more about load sharing with both mechanical fasteners and adhesive bonded construction under static and dynamic loading conditions. Tests conducted at WES (Ref 1) and SSI (Ref 2) considered simple beams consisting of three joists with flooring attached. Lateral load sharing between joists would not be evident in such tests. More complete floor assemblies are required for this purpose.

3. Technical information on wood properties under dynamic loading conditions needs to be developed.

4. Efficient and economic test methods need to be developed for the testing of as-built and upgraded floor systems in real structures (structures slated for demolition for example) to collapse when subjected to static and dynamic loads.

2. TEST STRUCTURE

The test structure (Figure 1 and 2) was acquired from the Department of the Interior, Indiana Dunes National Lake Shore. This was one of several buildings in the area slated for demolition and was furnished to IIT Research Institute (IITRI) for the test program. Building selection was influenced by test requirements and availability within the time constraints of this study. Ideally, the test structure would be of "typical" construction for a single-family residential dwelling in this geographic area. A wood joist floor over a full basement was an essential requirement. Wood joists, girder and columns of "typical" size, span and spacing was also a requirement. Practical considerations included manageable size to facilitate manual loading, and reasonable access for truck traffic and the instrumentation van.

The building selected for the test had many of the desirable characteristics. It was a relatively small, single-family dwelling with a wood joist floor over a full basement. Inside dimensions of the basement in plan were 34 ft 7-½ inch x 18 ft 5-¼ inch, see Figure 3. The clear height, top of basement floor slab to bottom of joist was 6 ft 10 inch. Up to about joist 14 from the north end of the building, see Figure 4, the floor consisted of 1-inch-thick and 3-inch-wide tongue and groove boards. Beyond, about joist 14 the floor consisted of 2 inch tongue and groove flooring, i.e., two layers of 1 inch boards, 3 inch wide. The bottom layer was nailed to the joists using 8d nails. The top layer in turn, was nailed to the bottom layer using 6d nails. The joist arrangement is shown in Figure 4. Joists were 2 x 6 inch with a mean center to center spacing of 24.12 inch. The joists were supported by the longitudinal, concrete block basement walls and a timber girder running parallel to these walls. The girder was in turn supported on three concrete block columns, one steel column, and the transverge basement walls. Column 2, see Figure 4, did not exist in the original construction. The species of the material was identified (Ref 4) as jack pine. The building

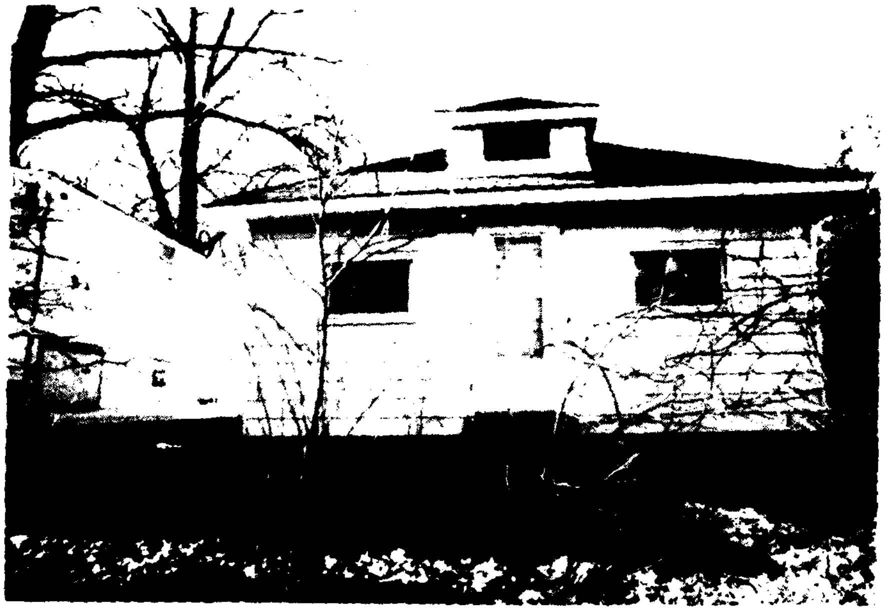


Figure 1. Front of Test Building

Figure 2. Rear of Test Building

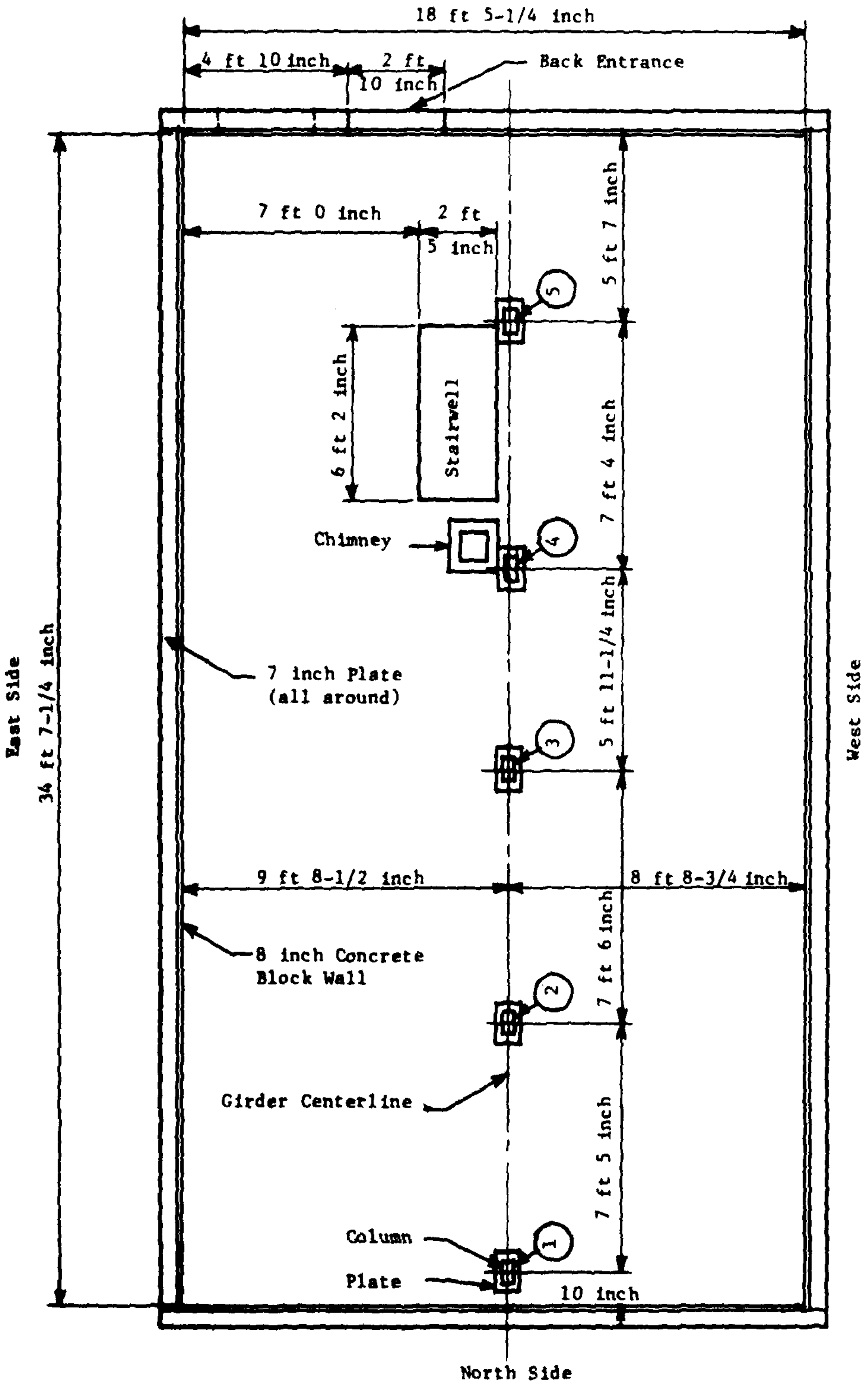


Figure 3. Basement Plan

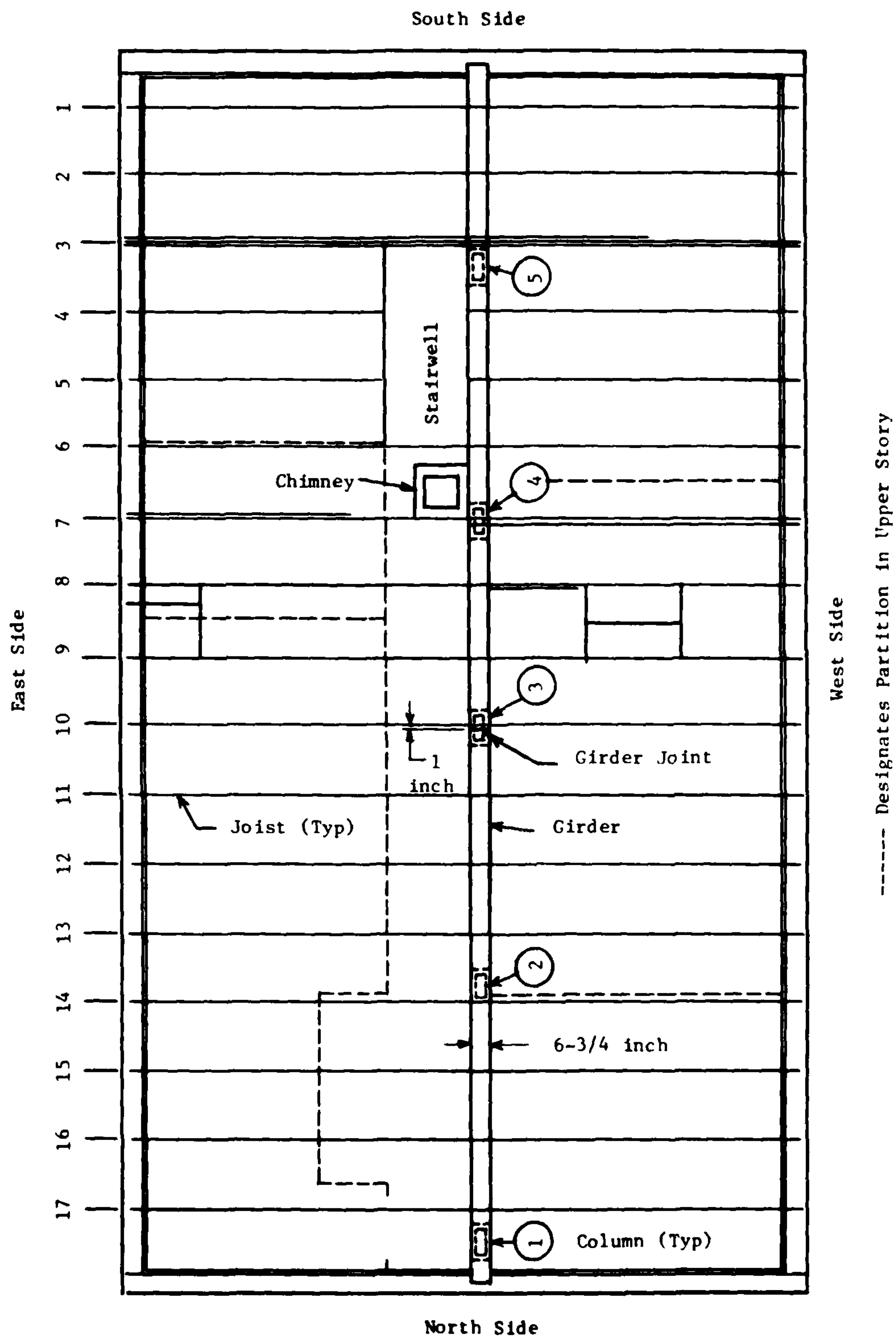


Figure 4. Joist, Girder, and Upper Story Partition Layout

was received in good, habitable condition and required only nominal effort to prepare it for the first load test.

The construction of the test building could not be described as typical for a single-family dwelling in this geographic area by current standards. Atypical characteristics that influenced the experimental results are briefly described.

1. At two locations the floor had been strengthened by adding parallel members to existing joists (see Figure 4). Cross members had been added to two other locations.

2. As indicated earlier, the floor thickness was not uniform for the entire floor. The front (north) portion of the building (see Figure 4) where the floor was of a single layer, was apparently an open porch which was later enclosed.

3. The girder was a composite formed by using two 2 x 6 inch boards to encase three 2 x 4 inch boards. A 0.25 inch piece of plywood was used as a filler. The girder was approximately 6-3/4 inch wide and 5-½ inch deep.

4. The joists (2 x 6 inch) were smaller than current practice (2 x 10 inch, 2 x 12 inch), and their spacing of 24 inch was longer than current size of 16 inch in this area. Some of the joists were notched to accommodate installation of wiring and electrical fixtures.

5. The spacing (about 15 ft) between the first two columns (columns 1 and 3, see Figure 3) was unusual in comparison with the other column spaces. To make the spacing more uniform, column 2 was installed in the test preparation stage. Also, the concrete block columns and the steel pipe column were removed and replaced with what we considered to be conventional timber columns. The new columns consisted of four 2 x 4 inch boards nailed together with plates made of 2 x 4 inch boards top and bottom.

6. The interior partitions in the upper story were constructed using 1 x 6 inch tongue and groove boards over 2 x 4 inch studs, both sides. The wall studs were at 24 inch centers.

3. LOAD TESTS

3.1 Methods for Applying the Test Load

Several methods for applying the static test load were briefly compared in terms of time and material requirements and cost. These methods (Ref 5) included hydraulic jacks, water, vacuum, and deadweight (sandbags, concrete block).

The use of hydraulic jacks would have required removing the upper story and building a reaction structure over the floor against which the jacks would act. This method was abandoned mainly due to the costly reaction structure that would have been required.

The unit weight of water (62.4 pcf) is relatively low so that large quantities are required as compared to more dense materials. The use of water as the load medium would have required sealing the upper story to prevent leakage and reinforcing the peripheral walls to support the lateral water pressure. Due to the depth of water required, it would have been necessary to seal the windows. The major difficulty with this method was the fact that water in large quantity was not readily available in the area and would have to be hauled in by a special truck. This method was therefore not entirely feasible for this location.

The vacuum technique involves evacuating the air from the space underneath a building element so that atmospheric pressure may act against the surface that is to be loaded. The space beneath the floor must be made into a sealed chamber in order for this method to work. This usually involves the construction of bulkheads surrounding the area to be treated. These bulkheads must be designed with ample safety factors to resist the atmospheric pressure that is applied. The floor or surface below, of course, will be loaded in the opposite direction, and provisions must be made to assure against failure at this surface. A great deal of study must be given to the method of sealing because an effective seal is most important to the proper performance of this loading method.

The vacuum technique is desirable since it allows for excellent precision of measurement and control of load. Its application to the test structure was explored in some detail. To implement it, it would have been necessary to remove the upper story and to seal the basement against air leakage. It would also have been necessary to shore the concrete block basement walls which protruded about 1 ft above grade. Pumps capable of evacuating the air from the basement at the required rate are available from manufacturers. This test method was not used due to the cost required to prepare the building for the test and the cost of rented equipment.

Sandbags were considered only briefly. It would have been necessary to purchase the bags and the sand. The filled bags would have to be filled, weighed, marked and stored in a place where they would absorb the least moisture prior to application on the test structure. This preparation prior to loading was thought excessive both in time and labor requirements.

Solid concrete block had approximately the right density and because of its size and weight could be placed on the structure fairly quickly by three or four individuals. It was a stock item at a local manufacturer and could be purchased in required quantities. The truck that delivered the block has a crane which could be used to move the pallets as required. One other advantage was that the manufacturer agreed to purchase back any undamaged block.

Thus the test load consisted of solid concrete blocks with nominal dimensions of 8 x 8 x 16 inch. Actual dimensions were 7.625 x 7.625 x 15.625 inch. The average weight of the block was 60.17 lb.

Disadvantages in using concrete block load on a large structure are the following:

(a) When large unit weights are required then care must be exercised to avoid or at least minimize "arching" of the block over the test structure. When arching occurs, the load is not uniform.

(b) If adequate shoring is not provided, damage to other parts of the structure or to instrumentation can occur.

3.2 Types of Load Tests

To maximize the data that could be obtained from the test structure it was decided to conduct two load tests.

(a) Load test of as-built floor

(b) Load test of expediently upgraded floor

The objective of the first load test was to determine the ultimate, uniform load static strength of the as-built floor system. That portion of the floor between the north wall and joist number 9 (see Figure 4) was used for this purpose.

The objective of the second load test was to determine the ultimate, uniform load strength of the floor system when expediently upgraded. Upgrading consisted of a studwall located halfway between the wall and the girder in each of the two spans and between the south wall and joist number 9, see Figure 4.

3.3 Preparation of the Test Structure for the First Load Test

Preparation of the test structure for the first load test involved the following tasks.

1. Remove the bath tub located approximately in the area between joists 8 and 9 against the east wall of the house (see Figure 4) and fill in the opening with plywood.

2. Remove electric conduits, where necessary.

3. Remove concrete block columns (items 1, 3, and 5, Figure 4) and the steel column (item 4, Figure 4) and replace them with timber columns with plates top and bottom. The new columns consisted of four 2 x 4 inch boards nailed together. The plates were 6-inch-long 2 x 4 inch boards. Replacement of columns was a safety measure because the concrete block columns were somewhat crooked and the mortar between the joints had partially deteriorated. The steel column was replaced for the sake of uniformity. It was felt that the existing columns did not represent

typical construction in this area and therefore their replacement was justified.

4. Provide a column between columns 1 and 3 (see Figure 4). The basement did not have a column at that location. It was felt that this was not typical and a column (item 2, Figure 4) was put in.

5. Build a studwall halfway between the wall and the girder under each of the two spans, with approximately a 3 inch clearance between the bottom of the joist and the top of the studwall. The purpose was to provide a "catcher" which would allow the floor to fully fail (break), but would prevent it from falling into the basement and possible damaging instrumentation.

6. Mark a grid on the floor for placing the block.

3.4 Displacement Measurement, First Load Test

Two parallel methods were used to measure the relative displacement of the floor system, i.e., linear potentiometer transducers and visual measurements. Locations where measurements were made are identified in Figure 5.

The displacement transducers were Computer Instruments Corp Type 113. These units have a resistance of 100 kohm and a linear range of 2.125 inch. The wiper arm of the transducer was spring loaded as shown in Figure 6. This gauge assembly was placed on the basement floor under the measuring locations. A 7.5 lb steel weight was suspended from the measuring location by using 6 gauge (0.016 inch) diameter music wire. Number 2 steel screw eyes were used to attach the music wire to the measuring locations. The weight rested on the spring-loaded wiper arm, thus providing an electrical output proportional to the relative displacement of the measuring location. The output of 38 displacement transducers was recorded on a Fluke 2240B Data Logger.

The visual measurements were accomplished by attaching a plastic tape rule (scale) to the music wire. Plastic tape rules were also attached at various locations on the concrete block basement walls to provide a fixed measuring reference. The tape

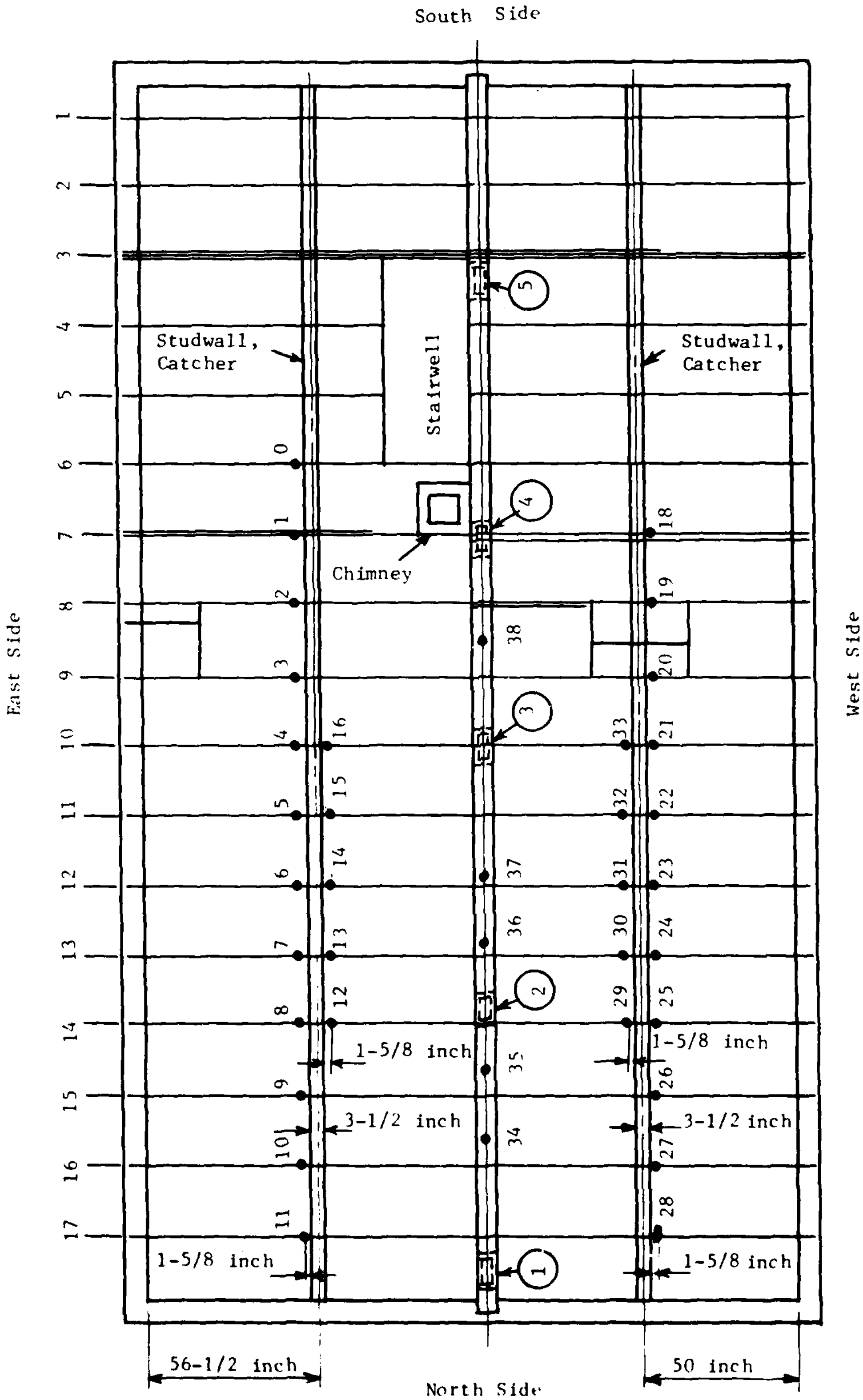


Figure 5. Deflection Measurement Instrumentation Layout, First Load Test

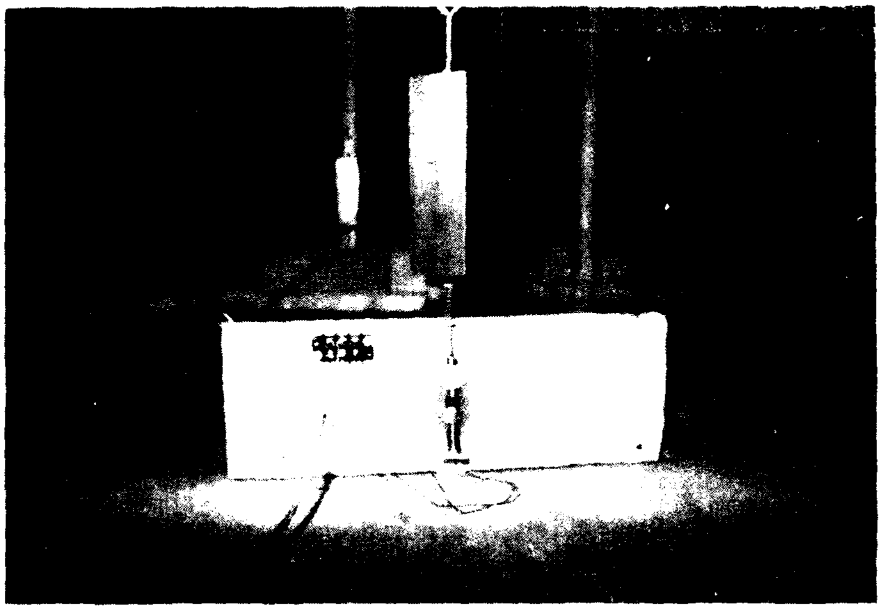


Figure 6. Potentiometer Setup

Figure 7. Studwall Catcher Showing Location of Potentiometers and Scales Attached to Potentiometer Suspension Wires rules were viewed by using a surveyor's level thus, providing appropriate redundancy in the measuring system.

Plastic tape rules were attached as shown in Figure 7.

3.5 First Load Test

As indicated earlier, the floor was marked in a grid pattern (see Figure 8) and the blocks were placed in a random distribution, thus providing a relatively uniform load during the loading process. The blocks were conveyed into the house using the method illustrated in Figure 9. The east and west sections of the test structure were uniformly loaded from joist number 9 to the north wall (see Figure 4). The sequence for this load test was as follows (see Figure A.2, Appendix A).

- The load was applied in equal increments up to a total load of 15 pallets (154.5 psf) in approximately 8.4 hours.
- The load of 15 pallets was maintained constant for 22 hours.
- The load was increased to 18 pallets (185.4 psf) and maintained constant for 12 hours
- The load was increased to 18-½ pallets (188 psf)

The load was applied in one-quarter pallet increments. One-quarter pallet consists of 15 blocks (~900 lb). Deflection readings were taken at the end of each pallet and every hour when the load was maintained constant. One pallet provided approximately 10.3 psf.

Failure was experienced approximately 6 hours after applying the eighteenth pallet. Cracks were observed in joists 14 and 15, see Figure 10 and 11. When the load was increased to 18-½ pallets, joists 13E through 17E failed. These failures are shown in Figures 12 through 16. Figure 17 shows the distortion of the wall plate on the north wall produced by the uplift of the girder end at that location.

With the exception of joist 14E, all of the other failures were flow (defect) related. Following is a brief description of the defects that contributed to the joist failure.

Figure 8. Grid and Initial Loading Arrangement

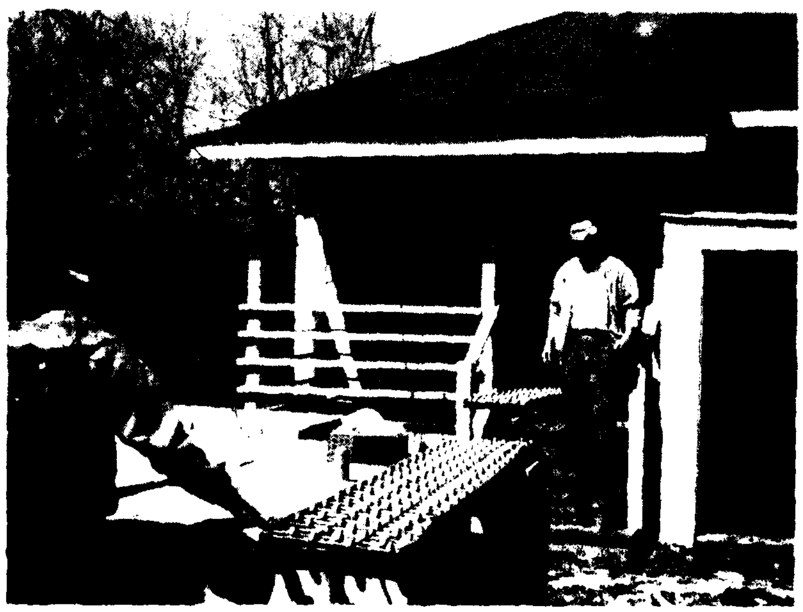


Figure 9. Concrete Block Pallets and Conveyor

Figure 10. Initial Failure of Joist 14E

Figure 11. Initial Failure of Joist 15E

Figure 12. Failure of Joist 13E

Figure 13. Failure of Joist 14E

Figure 14. Failure of Joist 15E

Figure 15. Failure of Joist 16E

Figure 16. Failure of Joist 17E

Figure 17. Distortion of Plate at North Wall

Figure 12, joist 13E - A knot through the joist approximately 1.25 inch diameter at the start of the crack. A second knot 12 inch along the crack line, approximately 1.5 inch diameter, extending 0.5 inch into the joist.

Figure 13, joist 14E had no visible flaw aside from the initial failure cracks shown in Figure 10.

Figure 14, joist 15E had a man-made notch to accommodate electrical wiring. The notch was about 0.8 inch wide and 0.8 inch deep and was located near midspan of the joist.

Figure 15, joist 16E had a knot near midspan about 1.65 inch diameter through the joist. Also a man-made notch for an electrical wiring junction box, 2 inch deep and 4 inch wide located 7 ft from the east wall.

Figure 16, joist 17E had a 1.5 inch diameter knot at the base of the joist, located 3 ft from the east wall.

Figures 18 and 19 show joist deflections as a function of joist position for indicated load levels. In Figure 19 a perturbation is centered about the position of joist 14W. This is attributed to the influence of the partition (wall) located above this joist in the upper story, see Figure 4. The wall acted in stiffening this joist and in transferring a portion of the load to the side walls of the upper story. From Figures 18 and 19 it is evident that the joists did not act independent of each other and some plate action due to the flooring was present.

Figure 20 shows average load-deflection curves for joists 11E through 15E and for joists 10W through 13W. Joists located at some distance away from the edges, i.e., north and south edges of the loaded portion of the test structure, were selected for this comparison. Due to shorter span, joists on the west side are stiffer than those on the east side. Deflections under constant load are evident at 154.5 psf and 185.4 psf.

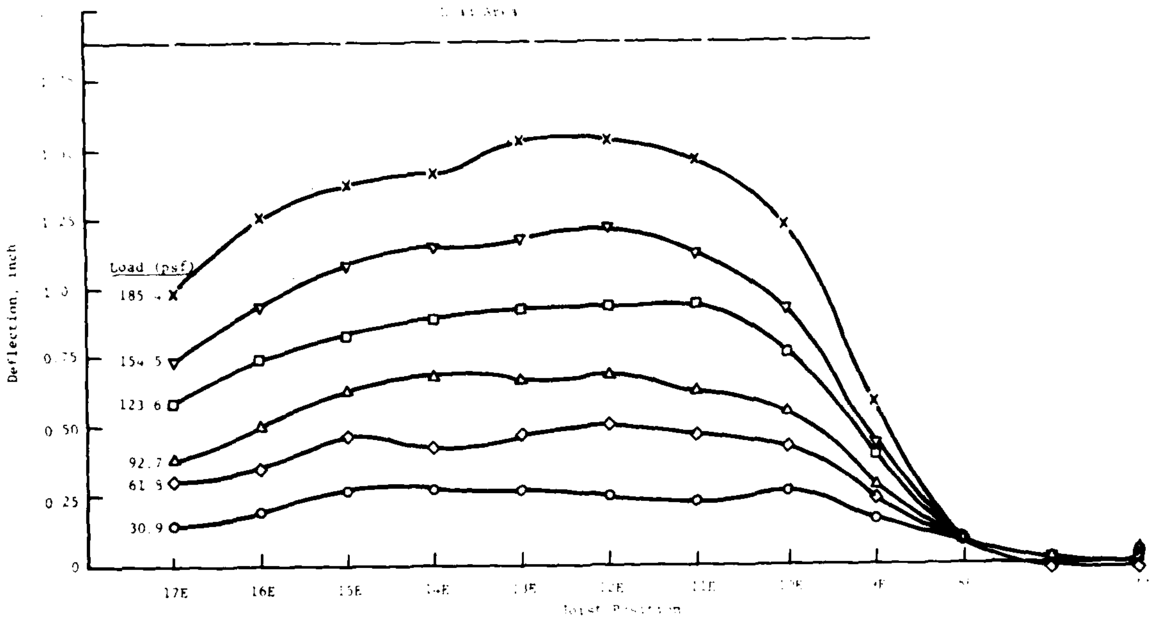


Figure 18. Deflection versus Joist Position for Indicated Load Levels - East Side, First Load Test

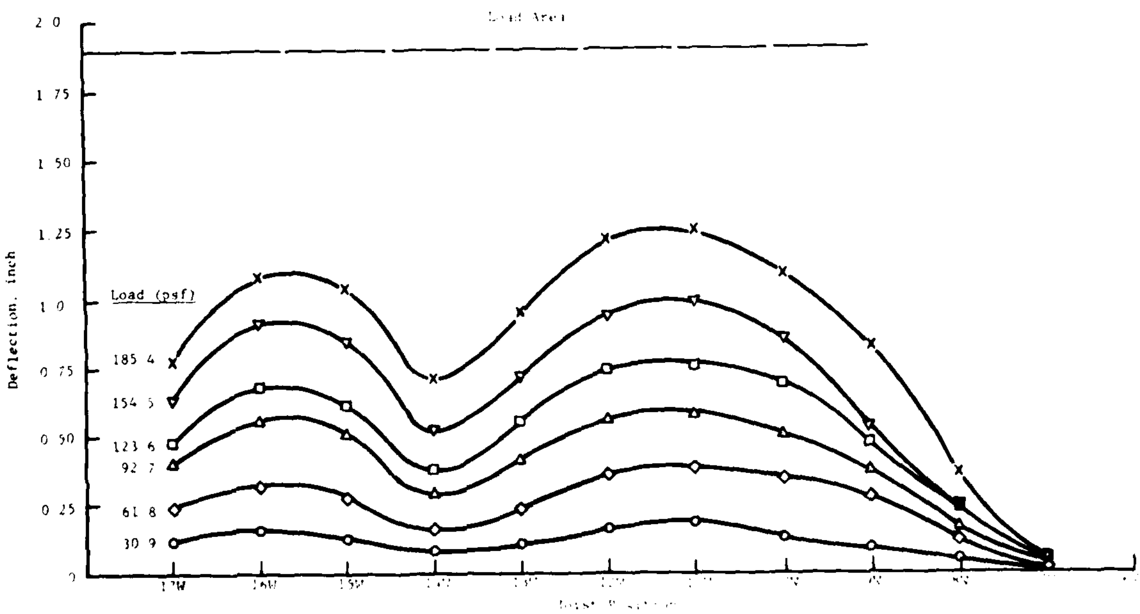


Figure 19. Deflection versus Joist Position for Indicated Load Levels - West Side, First Load Test

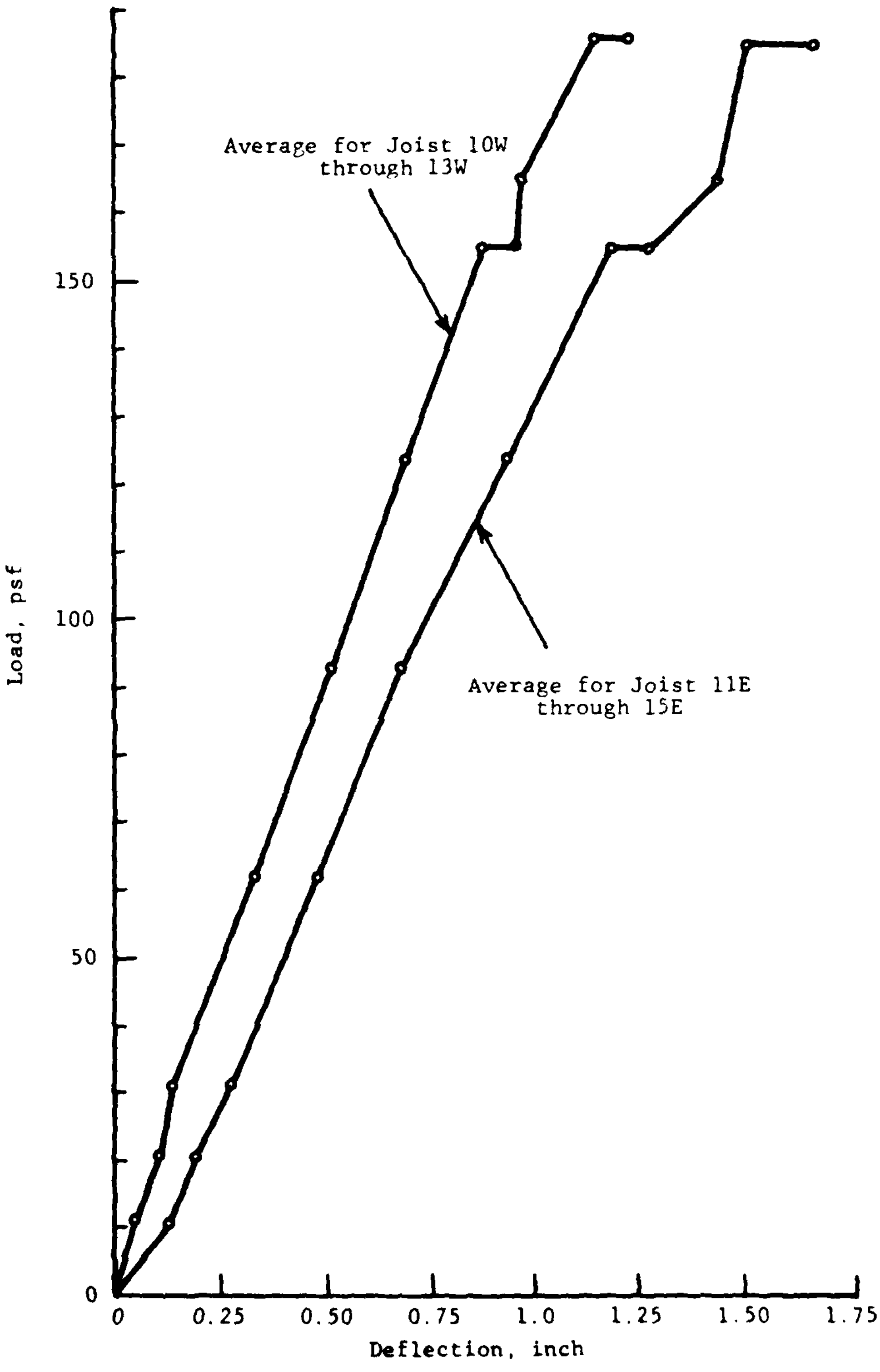


Figure 20. Average Load-Deflection Curves for Joists, First Load Test

For this load test, load-deflection curves for individual joists are given in Figures A.3(a) through A.3(j) in Appendix A. Deflection-time curves at constant load are given in Figures A.4 and A.5. Load-deflection curves for the girder are given in Figures A.8(a) through A.8(e).

3.6 Preparation of Test Structure for the Second Load Test

Preparation of the test structure for the second load test involved the following tasks.

1. Remove the stairs from the upper story to the basement and make the floor at the stairwell continuous.
2. Remove the upper story to facilitate loading the floor with the aid of a crane.
3. Provide a waterproof cover for the exposed floor.
4. Repair the previously tested portion of the basement for use as an observation area for this test.
5. Upgrade the floor over the basement by placing a studwall halfway between the exterior wall and the girder in each of the two spans.

The studwall expedient upgrading is shown in Figure 21. The timber wall in upgrading was spruce.

3.7 Displacement Measurement, Second Load Test

The method used for measuring floor displacements in the first load test was also used in the second load test. Displacement transducers and plastic tape rules were moved and installed at locations shown in Figure 22.

3.8 Second Load Test

The east and west floor sections were uniformly loaded from a line midway between joists 8 and 9 to the south wall (see Figure 22).

Figure 23 is a view of the floor and the expedient upgrading as implemented for this test. As a safety measure the two studwalls were braced with diagonal members. This photograph is a view directly toward the north wall of the basement. It also shows the filled-in stairwell, the girder and the timber columns which replaced the original concrete block columns.

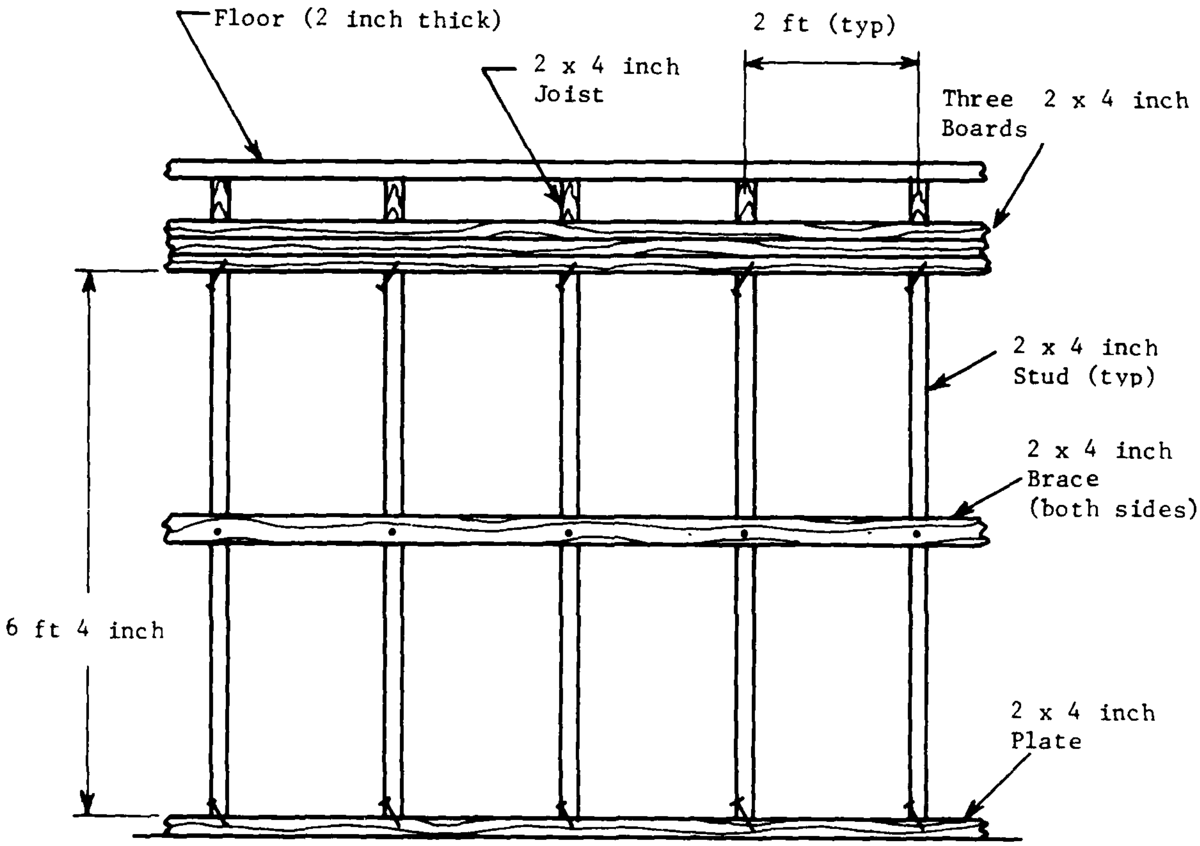


Figure 21. Expedient Upgrading

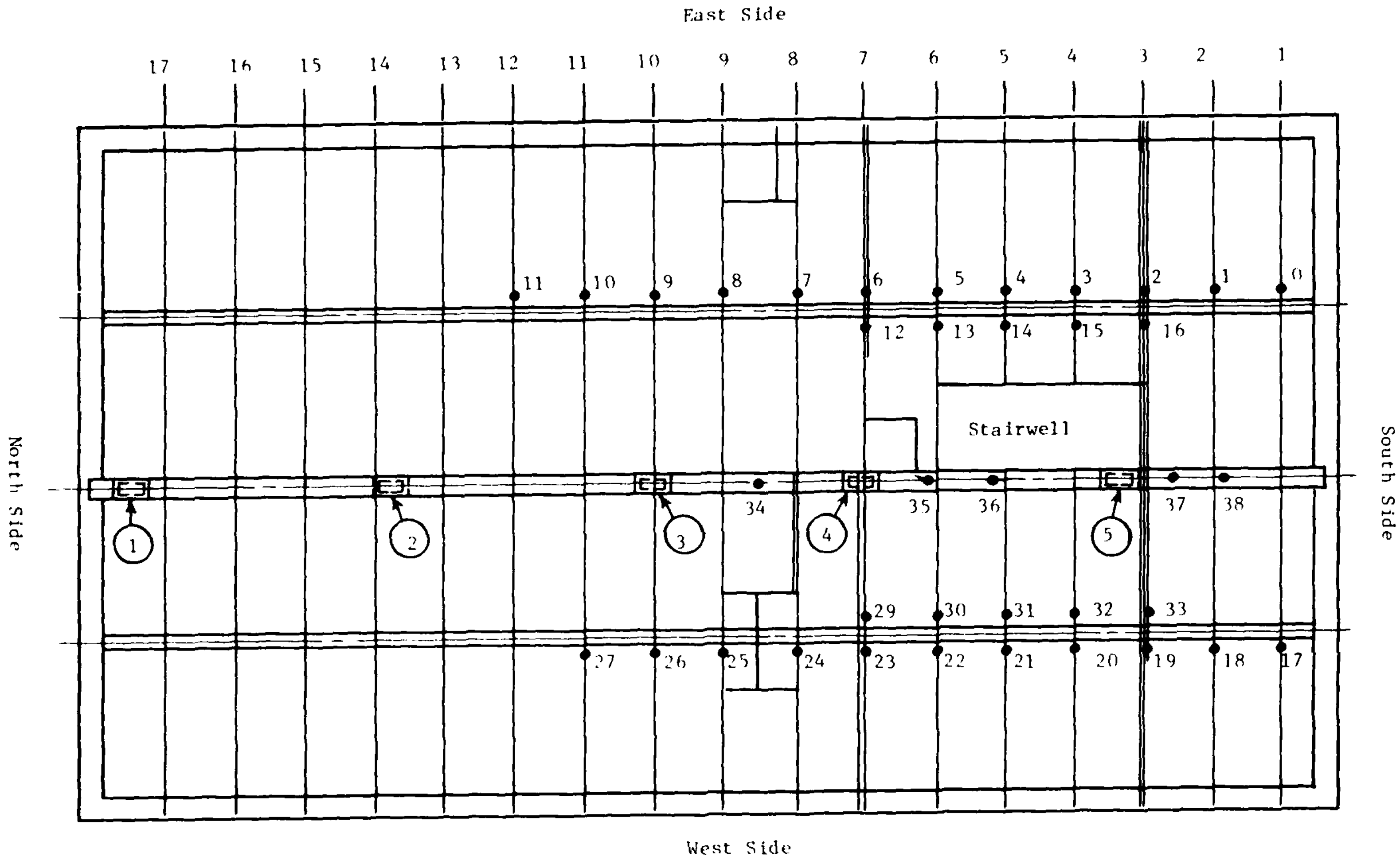


Figure 22. Deflection Measurement Instrumentation Layout, Second Load Test

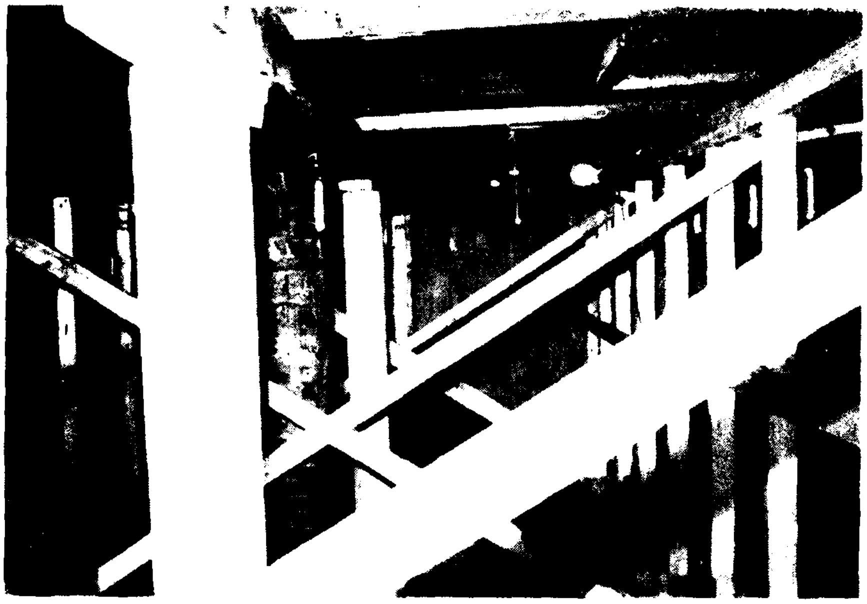


Figure 23. Diagonal Braces for Expedient Upgrading

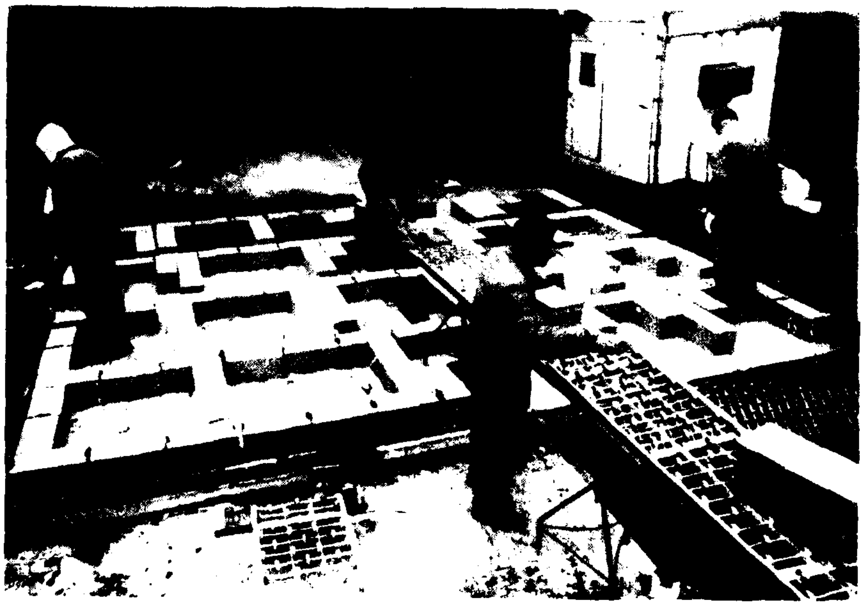


Figure 24. Start of Loading, Second Load Test

The loading sequence of the test structure is shown in Figures 24, 25, 26, and 27. Figure 24 shows the initial loading with the random distribution of the block and the method for conveying the block to the test structure.

Figure 25 shows the test structure loaded with 12 pallets of block, i.e., 43,200 lb. Figure 26 shows the test structure loaded with 42 pallets of block or 151,200 lb. The final load at which the loading was terminated is shown in Figure 27. The load consists of 47 pallets of block or 169,600 lb (559.3 psf) and is approximately 7 ft high.

The loading sequence was as follows, see Figure A.10, Appendix A. The floor was first loaded to 11 pallets (130.9 psf) and maintained constant for approximately 50 minutes. It was then loaded to 39 pallets (464.10 psf) and maintained constant for 12 hours. The load was then increased to 47 pallets (559.3 psf) and maintained for approximately 1 hour. The test was terminated because the safety of the personnel was in some doubt.

When the floor was inspected after the removal of the load, no visible or measurable signs of distress were found with the exception of local crushing (see Figure 28) where the joists were in contact with the studwall used for expedient upgrading. Some minor distortion of the girder and column plates at points of support of the girder with the columns was also observed.

Figures 29 and 30 show the joist deflections as a function of position at the indicated load levels. Figure 31 contains average load-deflection curves for joists 3 through 7 from the east and west sides of the floor system. These joists were chosen because they are sufficiently far from the edges so that the influence of edge effects is minimal. Deflection under constant load is evident at 464.1 psf.

Additional data from this load test are included in Appendix A. This includes individual joist deflections as a function of load, deflections as a function of time under constant load, and girder deflections as a function of load.

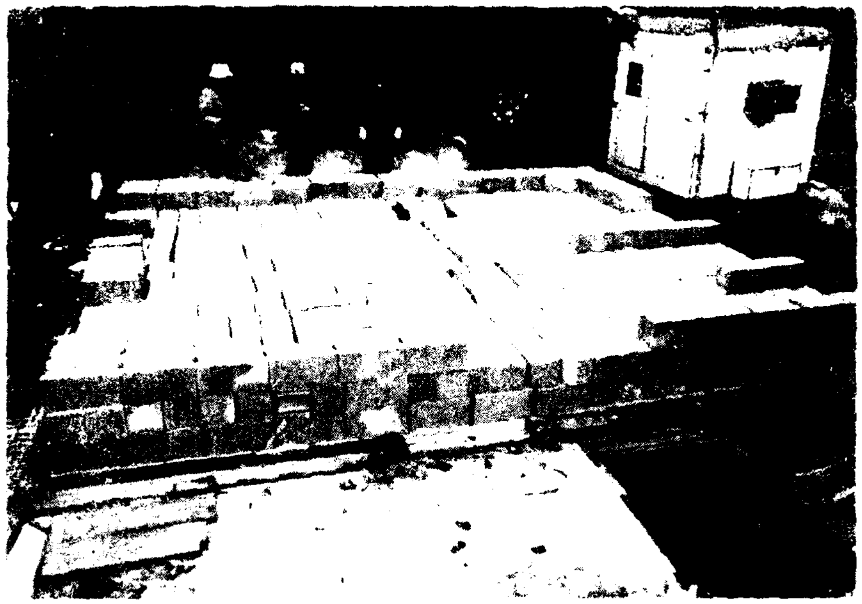


Figure 25. Load at 12 Pallets of Block, 43,2[illegible] lb

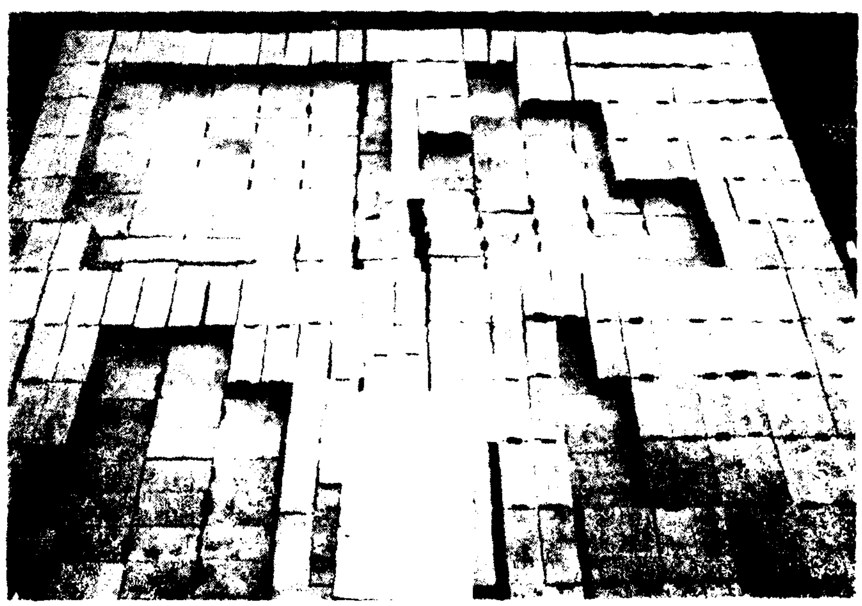


Figure 26. Load at 42 Pallets of Block, 141,[illegible] lb

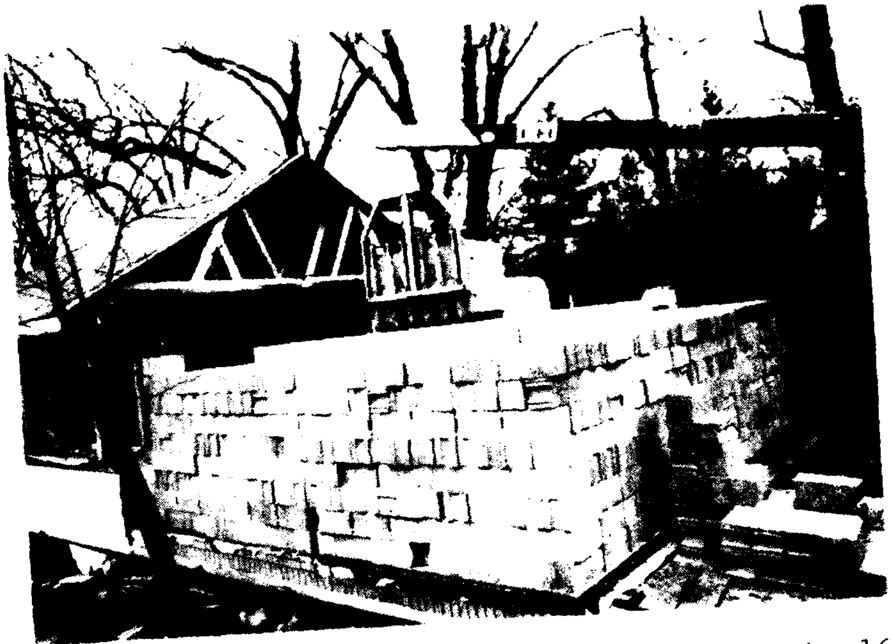


Figure 27. Final Load at 47 Pallets of Block, 169,600 lb

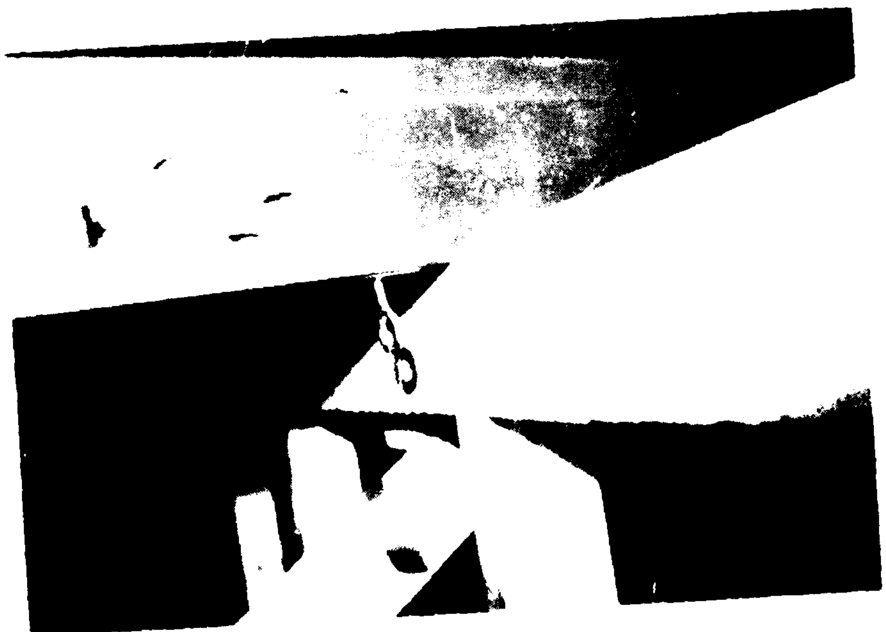


Figure 28. Local Crushing of Joist and Studwall

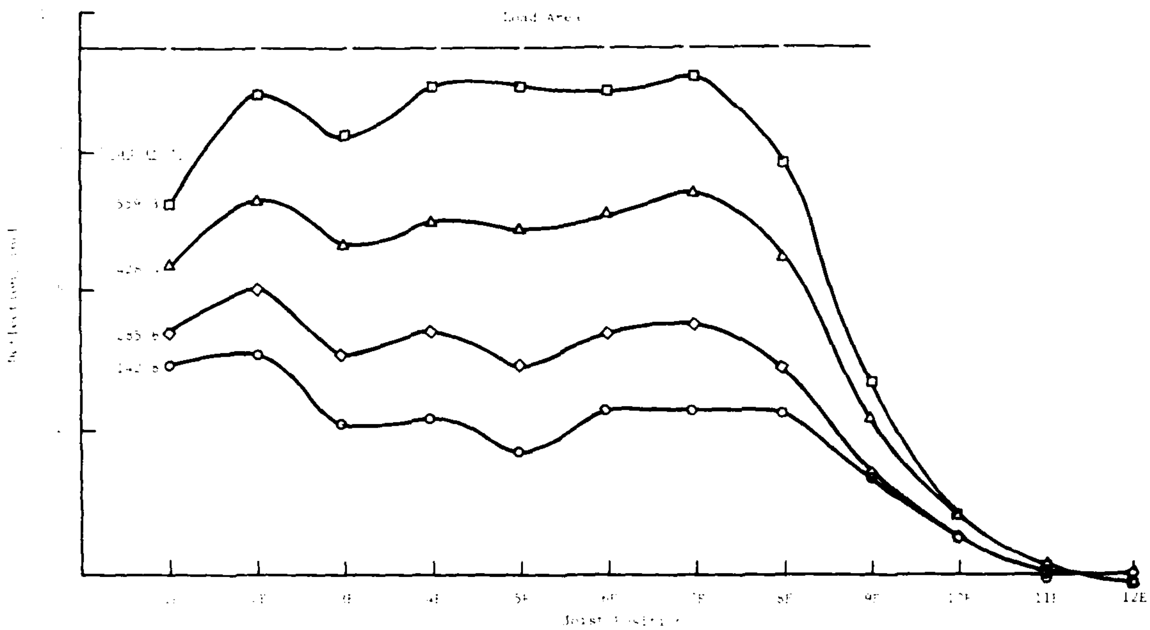


Figure 29. Deflection versus Joist Position for Indicated Load Levels - East Side, Second Load Test

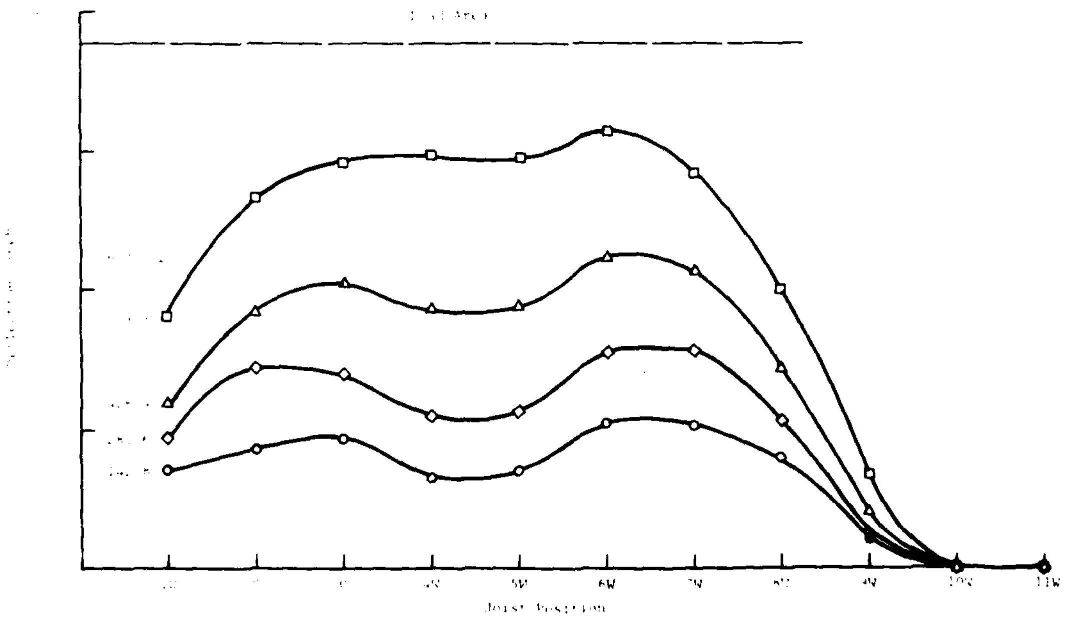


Figure 30. Deflection versus Joist Position for Indicated Load Levels - West Side, Second Load Test

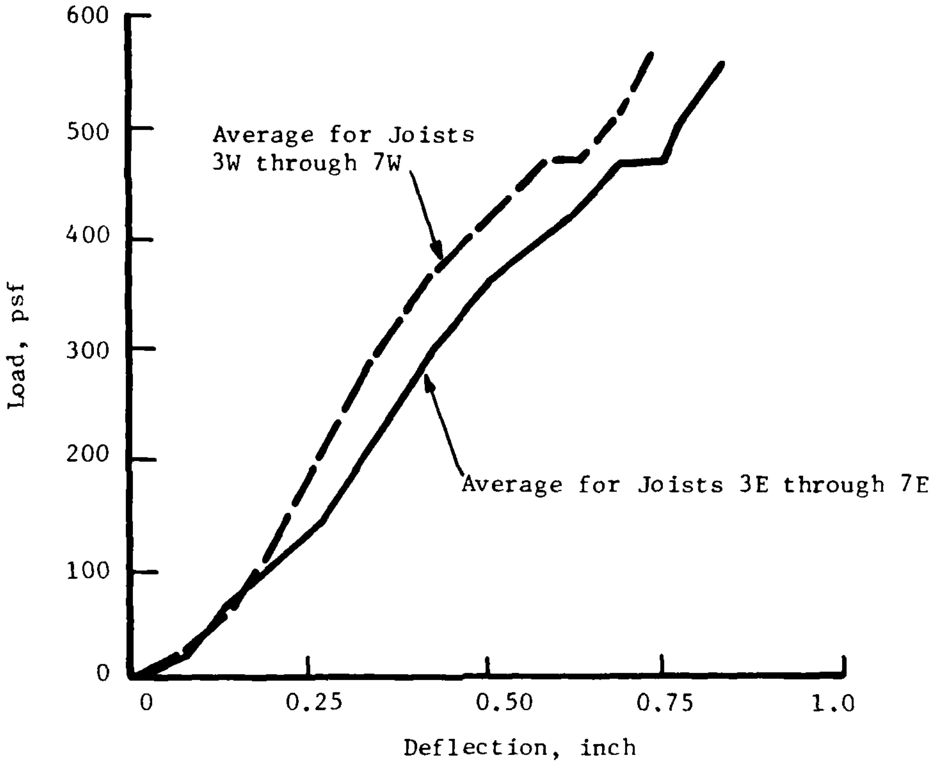


Figure 31. Average Load-Deflection Curves for Joists, Second Load Test

4. LABORATORY LOAD TESTS

To provide additional information on the behavior of the test floor, portions of the floor system were removed and tested in the laboratory. Floor samples were taken from the west side of the floor system. No failures were experienced in this portion during the in situ load tests. Three samples were removed. Each was 4 ft wide and approximately 10 ft long and consisted of two joists with the flooring attached. These samples are identified:

Sample 1: Joists 10 and 11 with a double layer of flooring.

Sample 2: Joists 12 and 13 with a double layer of flooring.

Sample 3: Joists 14 and 15 with a single layer of flooring.

The experimental setup used in conducting these tests is shown in Figure 32. This is a simple beam arrangement. The load was static and the loading medium was concrete block of the same type as used in the full-scale tests. Measurements were made of the midspan deflection of each joist. Deflection measurements were made by attaching a plastic ruler at the midspan of the joists and reading the relative deflection by means of a transit.

In the first sample both joists failed. Joist 10 failed at a point approximately 37 inch from midspan toward the left end of the test setup (see Figure 33). Joist 11 failed at a point approximately 34 inch from midspan toward the left end of the test setup. Failure did not appear to be flaw related. The failed conditions of joists 10 and 11 are illustrated in Figures 33 and 34 respectively. The concrete blocks shown in these two photographs supporting the joists near the points of rupture were used as catchers to prevent total collapse of the test setup after rupture. Failure load for this sample was 171 psf.

Sample 2 did not experience failure of both joists at the same time. Joist 13 failed first. Joist 12 failed two increments of load later. In this test failure was flaw related. The failed condition of joist 12 is shown in Figure 35. The failure load for this sample was 160 psf.

Sample 3 had one layer of flooring. It had man-made flaws a notch to accommodate wiring was located about 5 inch from midspan in each of the two joists. Figure 36 shows the initiation of a crack in joist 14 which occurred after block 56 was in place. After block 63 was added the crack appeared to increase. While pictures were being taken the crack propagated in both directions of the flaw and joist 14 failed. There was no sign of distress in joist 15. The failure load for this test was 98.2 psf. Load-deflection curves for three laboratory tests are shown in Figures 37, 38 and 39.

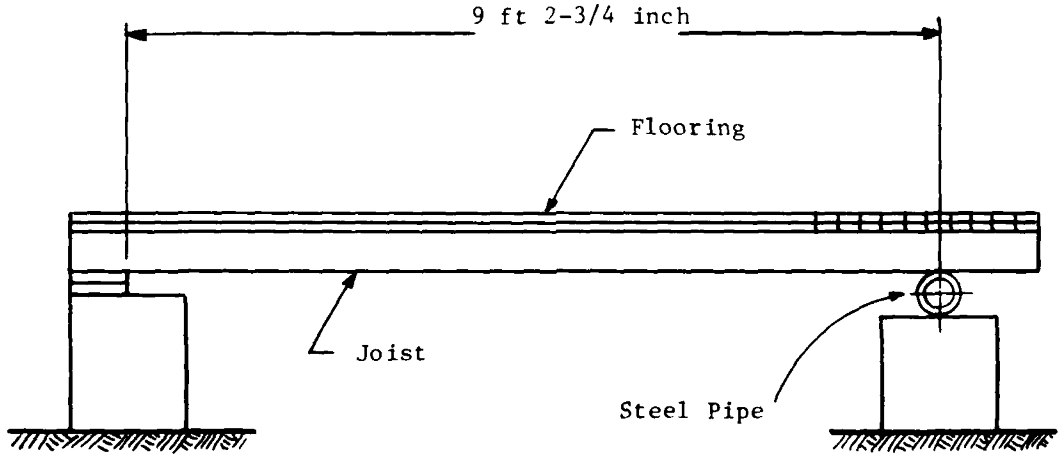


Figure 32. Experimental Setup for Laboratory Tests

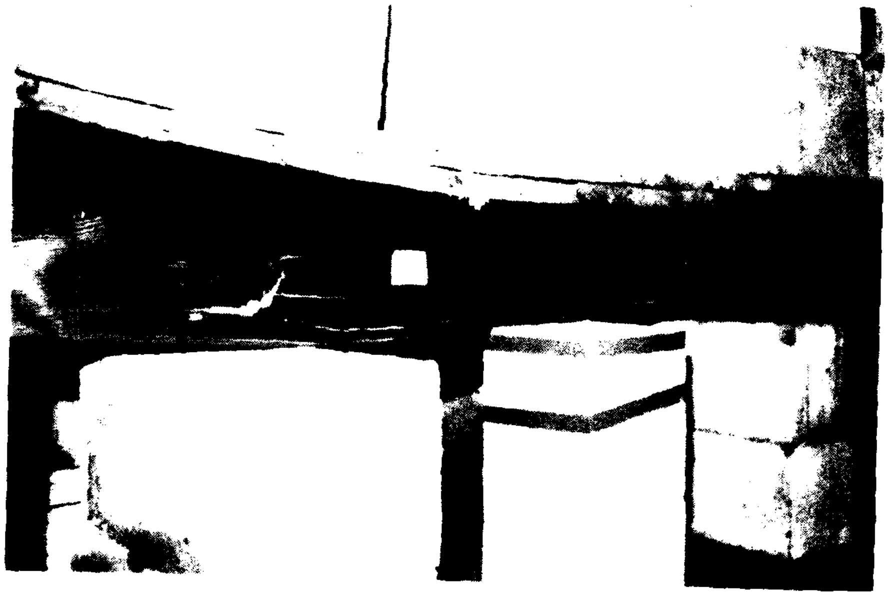


Figure 33. Failed Condition of Joist 10

Figure 34. Failed Condition of Joist 11

Figure 35. Failed Condition of Joist 12

Figure 36. Initiation of a Crack in Joist 14

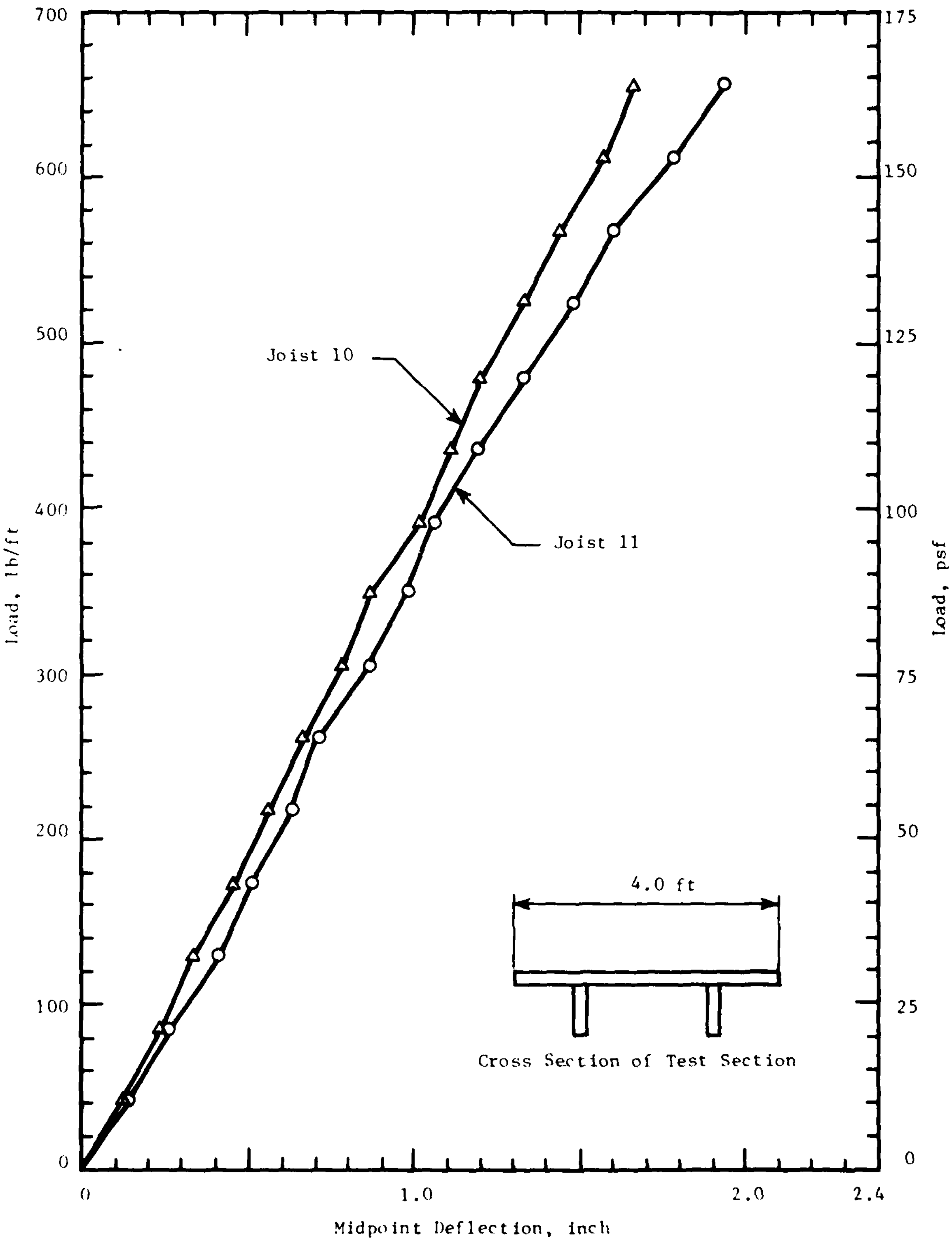


Figure 37. Variation of Load versus Midpoint Deflection, Joists 10 and 11

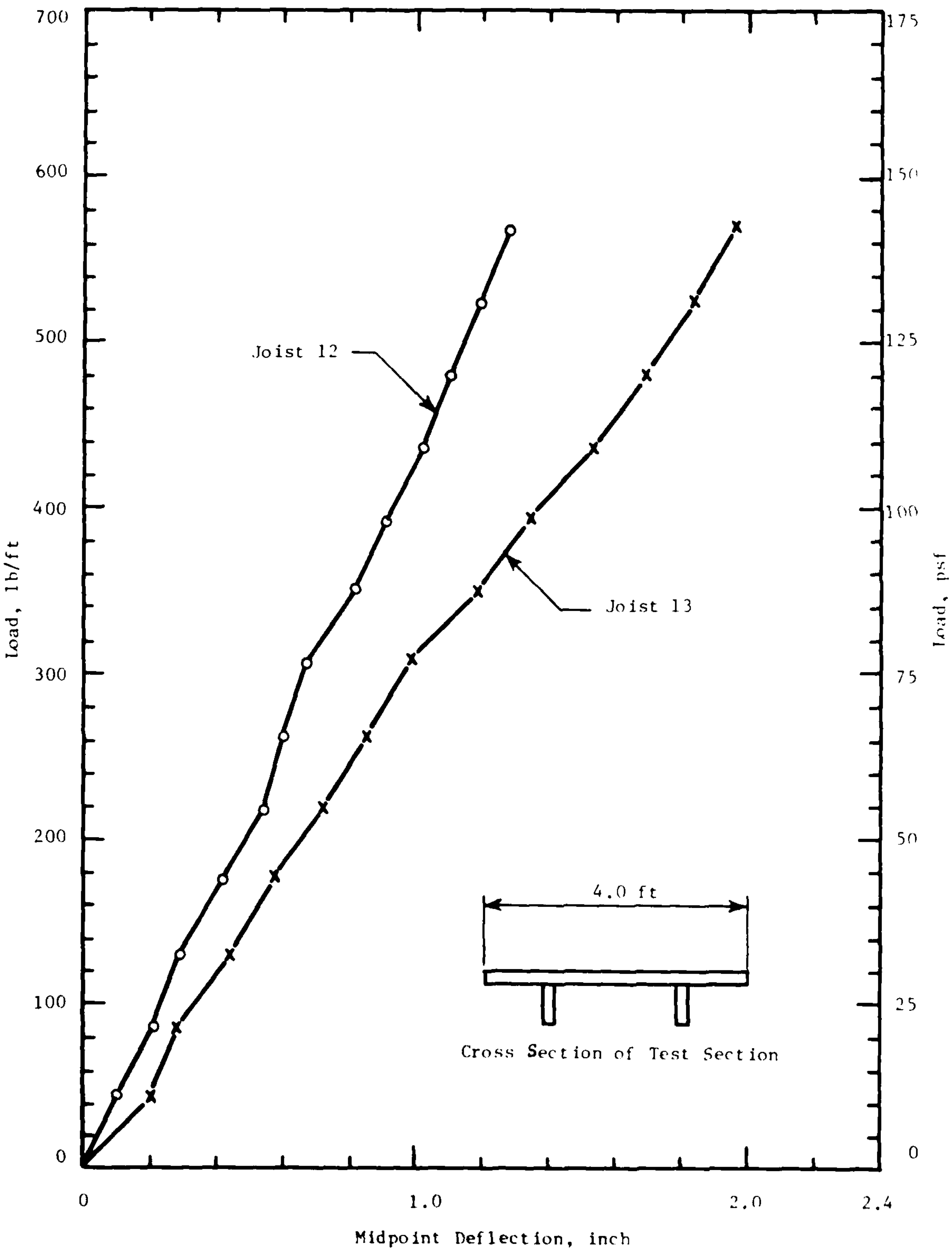


Figure 38. Variation of Load versus Midpoint Deflection, Joists 12 and 13

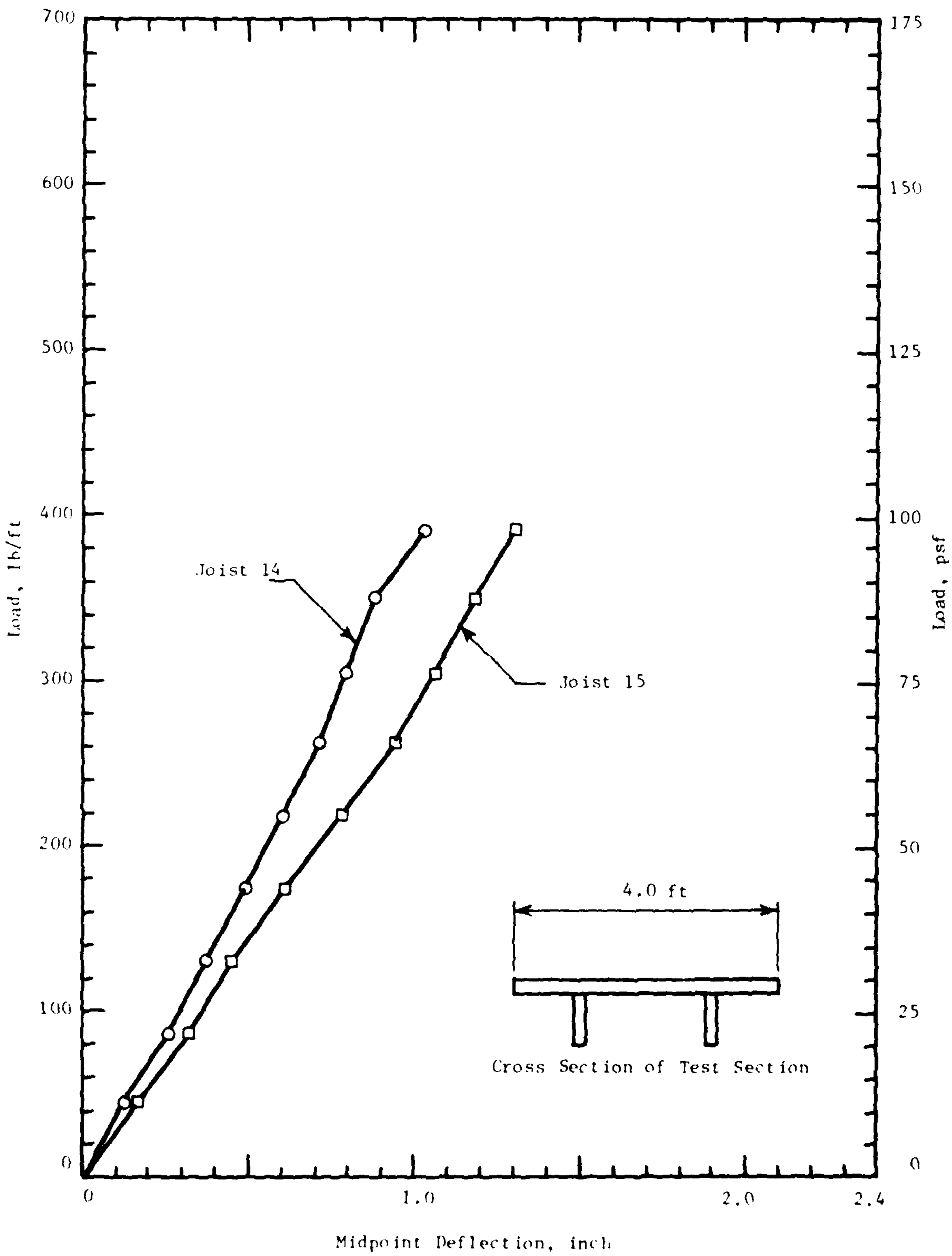


Figure 39. Variation of Load versus Midpoint Deflection, Joists 14 and 15

APPENDIX A

DEFLECTION MEASUREMENTS - FIRST AND SECOND LOAD TEST

A.1 First Load Test

For this load test, the deflection measurement instrumentation layout is shown in Figure A.1. Joist deflections were measured at positions 0 through 16 on the east side and at positions 18 through 33 on the west side. The measuring positions were 1-5/8 inch from either edge of the catcher. The catcher is 3-1/2 inch wide.

Girder deflections were measured at positions 34, 35, 36, 37 and 38. Positions 34, 37 and 38 are located halfway between the respective columns. Position 35 is halfway between position 34 and column 2, and position 36 is halfway between column 2 and position 37. Column spacing is given in Figure 3.

The loading sequence is shown schematically in Figure A.2. The floor system was first loaded in equal time and load increments up to a total load of 15 pallets (154.5 psf) in 8.4 hr. This load was maintained constant for 21 hr. It was then increased to 18 pallets (185.4 psf) and maintained constant for 12 hr. Failure occurred 6.5 hr after applying the eighteenth pallet, or 37.5 hr after the start of the test. Readings were taken after every pallet of applied load and every hour when the load was maintained constant.

Joist deflections for joists 8 through 17 are given in Figure A.3(a) through A.3(j). Increments shown in these curves are for every 10.3 psf of load, which is the same as the load increments. Figure A.4(a) shows deflection-time curves for joists 11 through 17 from the east and west sides of the test structure where the load of 154.5 psf was maintained constant for 21 hr. Figure A.4(b) shows the corresponding girder deflections at the five measuring positions. Figure A.5(a) shows deflection-time curves for joists 12 through 16 on the east and west sides of the test structure when the load of 185.4 psf was maintained constant for 12 hr.

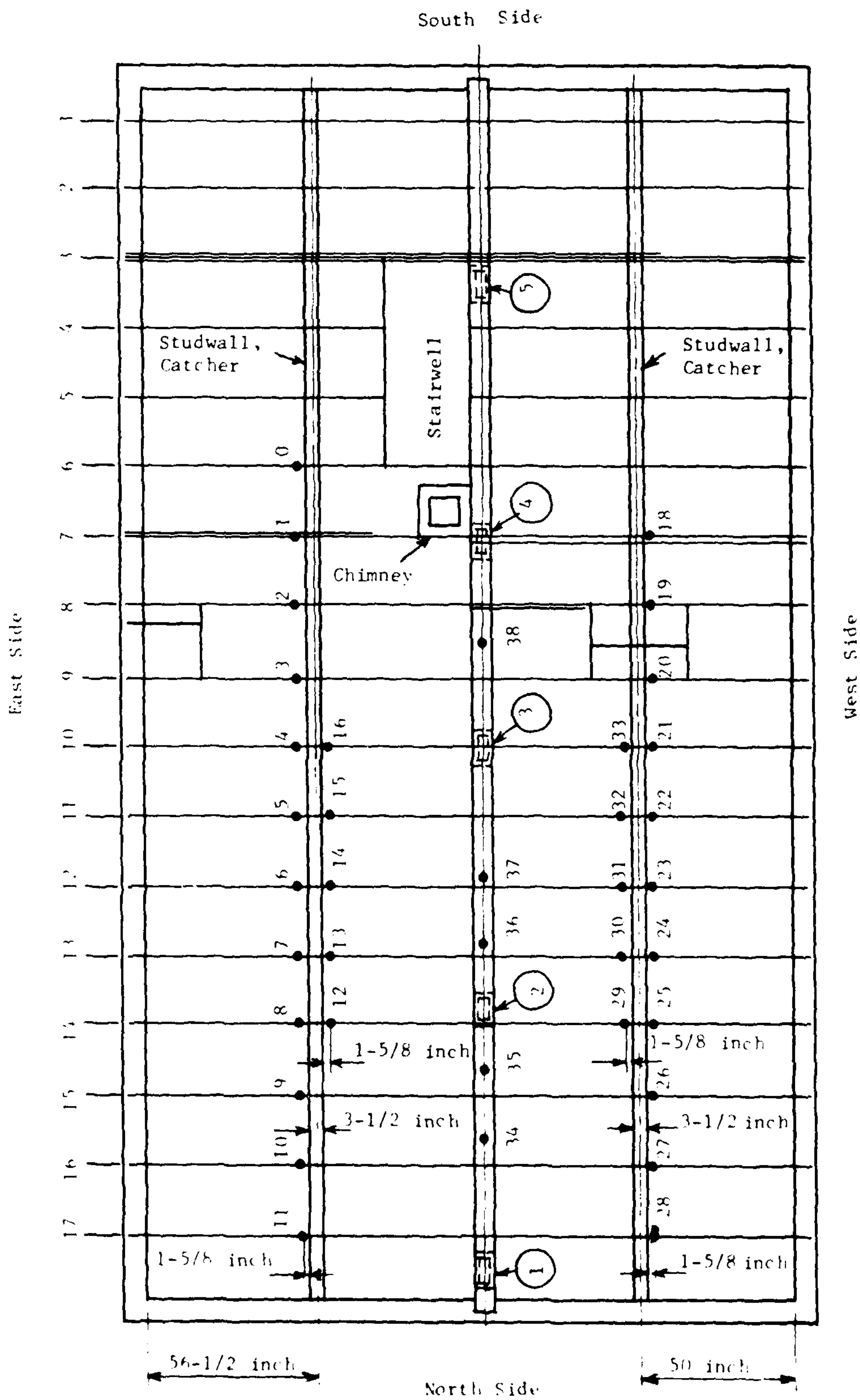


Figure A.1 Deflection Measurement Instrumentation Layout, First Load Test

Note: 1 Pallet = 10.3 psf

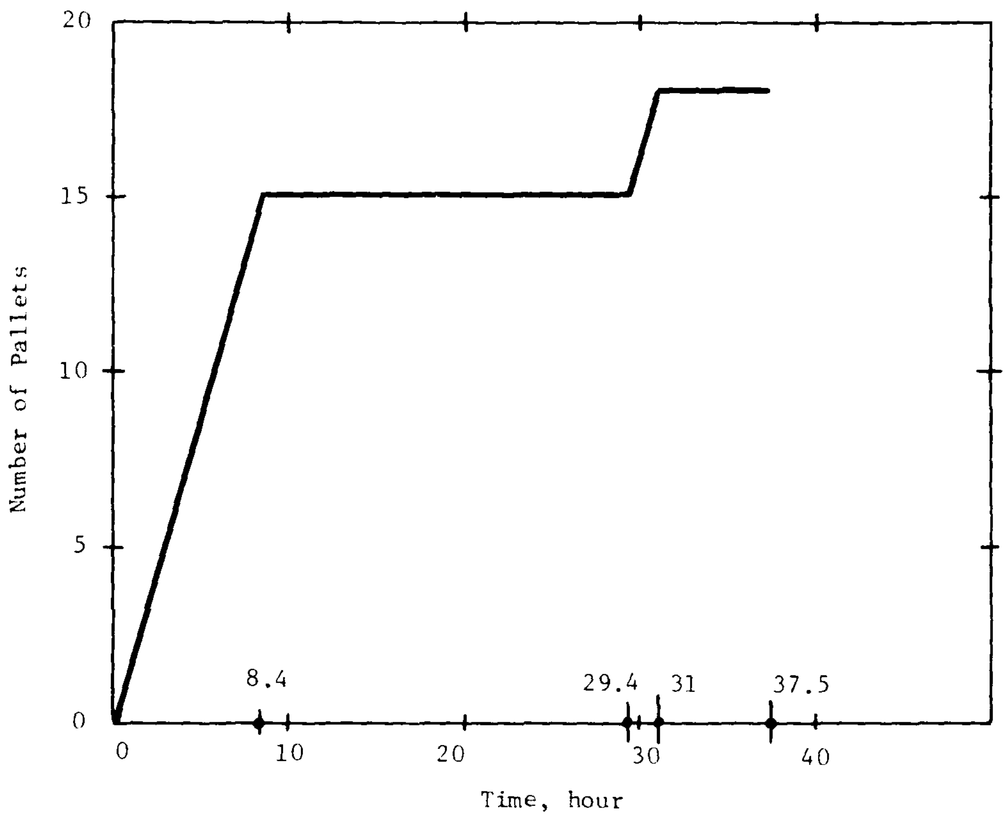


Figure A.2 Load-Time Diagram for the First Load Test

Figure A.5(b) shows the corresponding girder deflections at the five measuring positions. Note that failure was experienced approximately 6 hours after the application of the eighteenth pallet of block (see Figure A.5(a)). Joists 14E and 15E developed visible cracks. Further increase of load to 188 psf caused failure of joists 13E through 17E.

Joist deflections as a function of joist position are given in Figures A.6 and A.7 for the east and west sides of the test structure. Girder deflections at positions 34, 35, 36, 37 and 38 are shown in Figure A.8(a) through A.8(e).

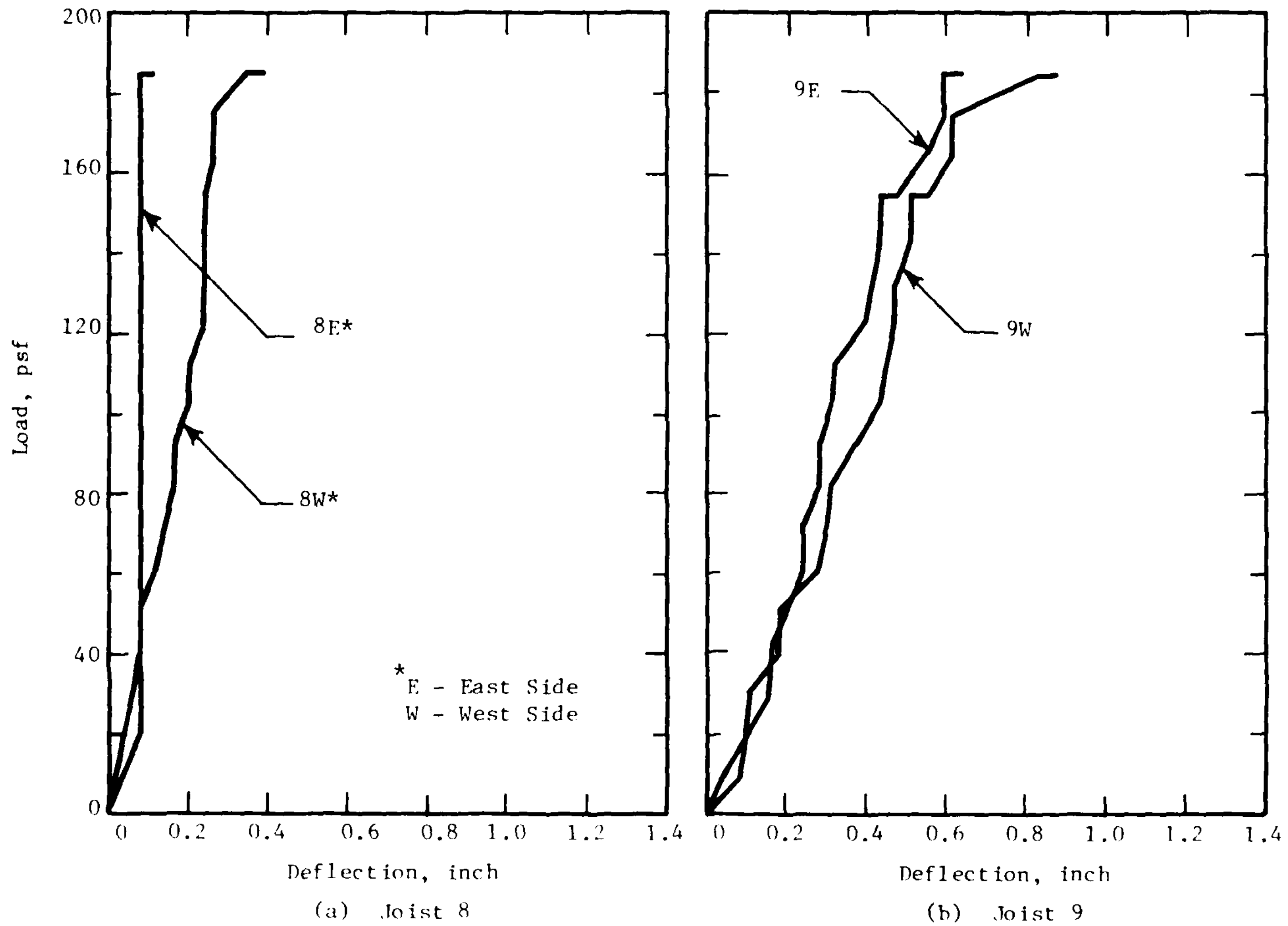
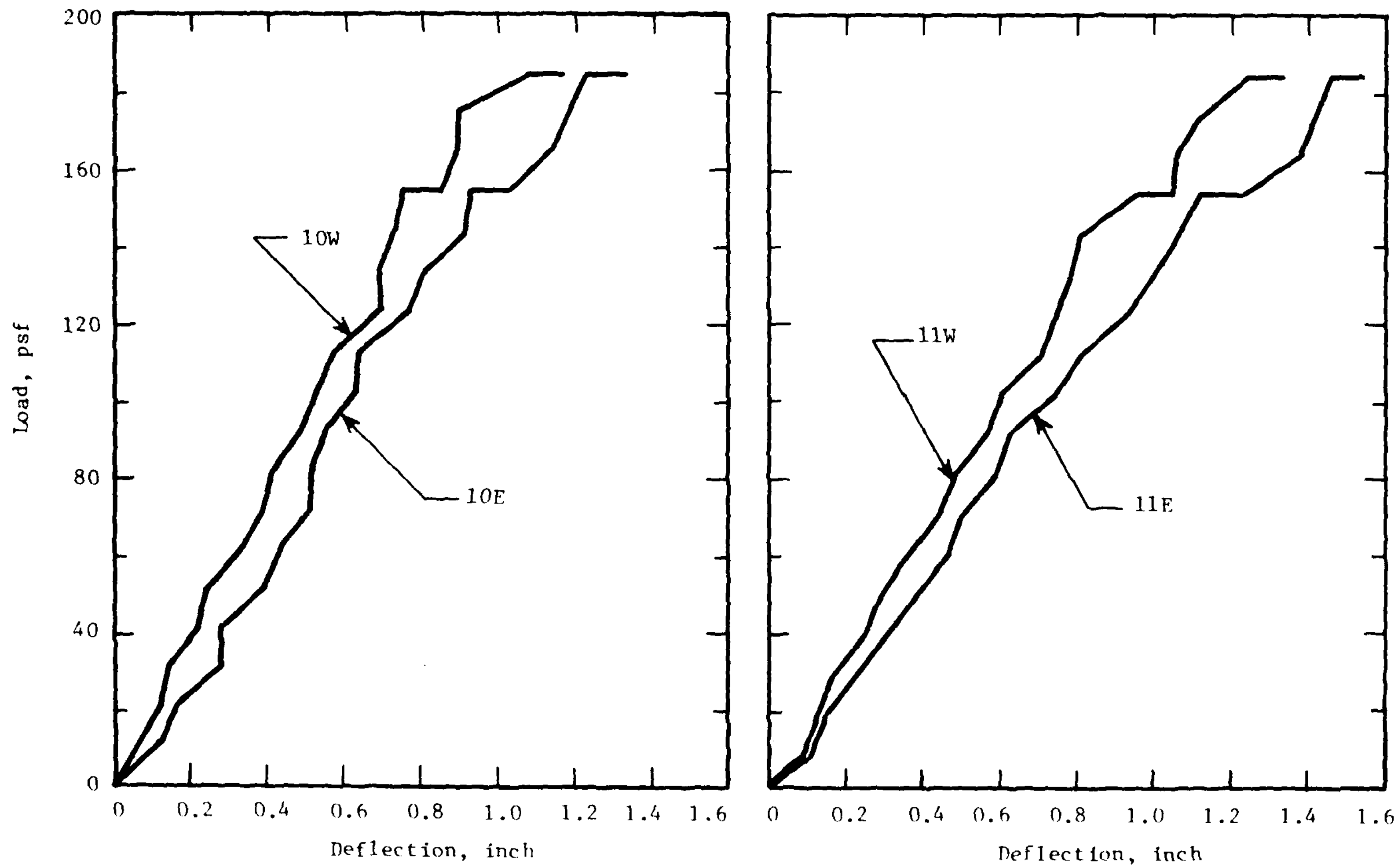


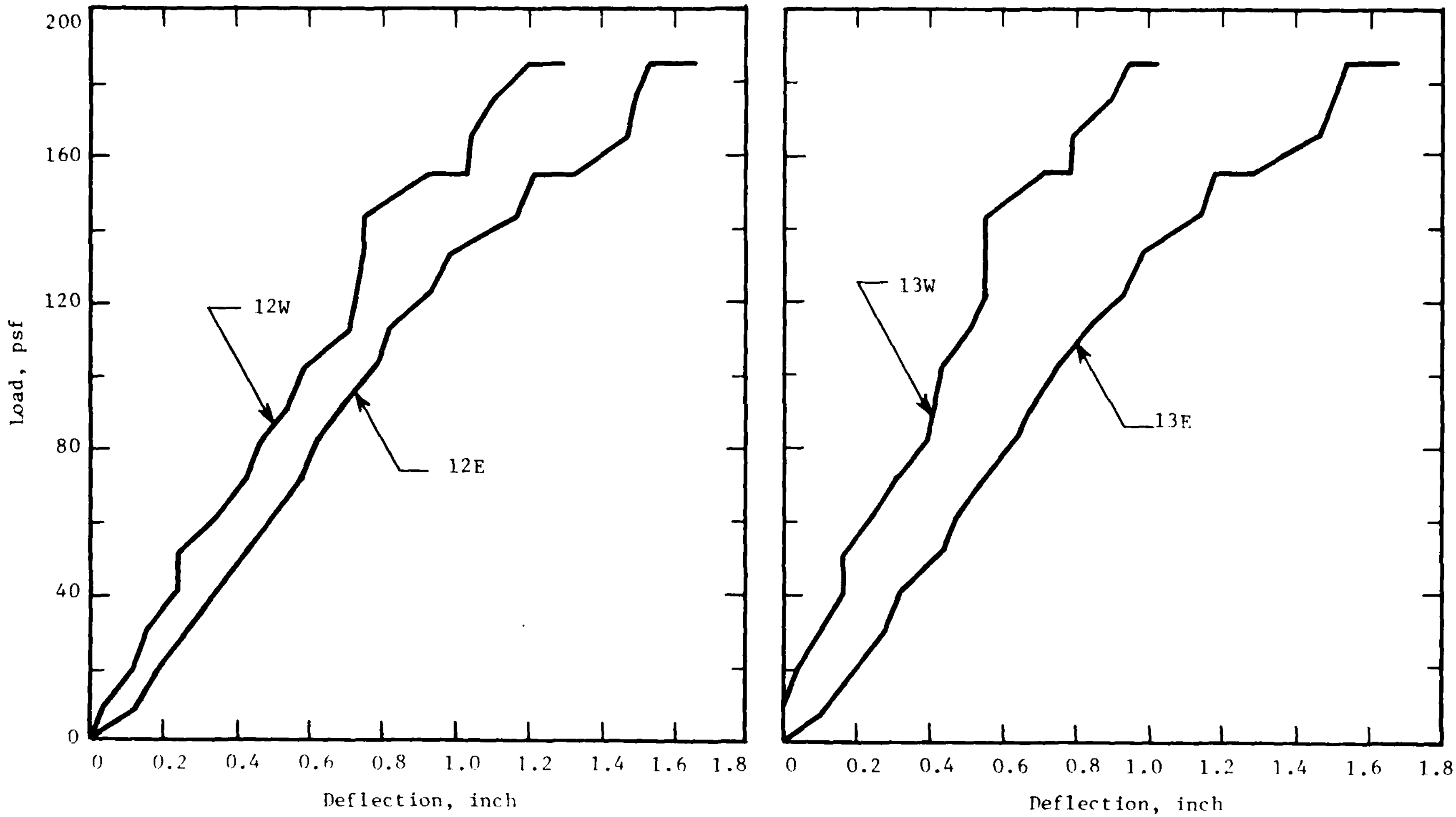
Figure A.3 Joist Deflections



(c) Joist 10

(d) Joist 11

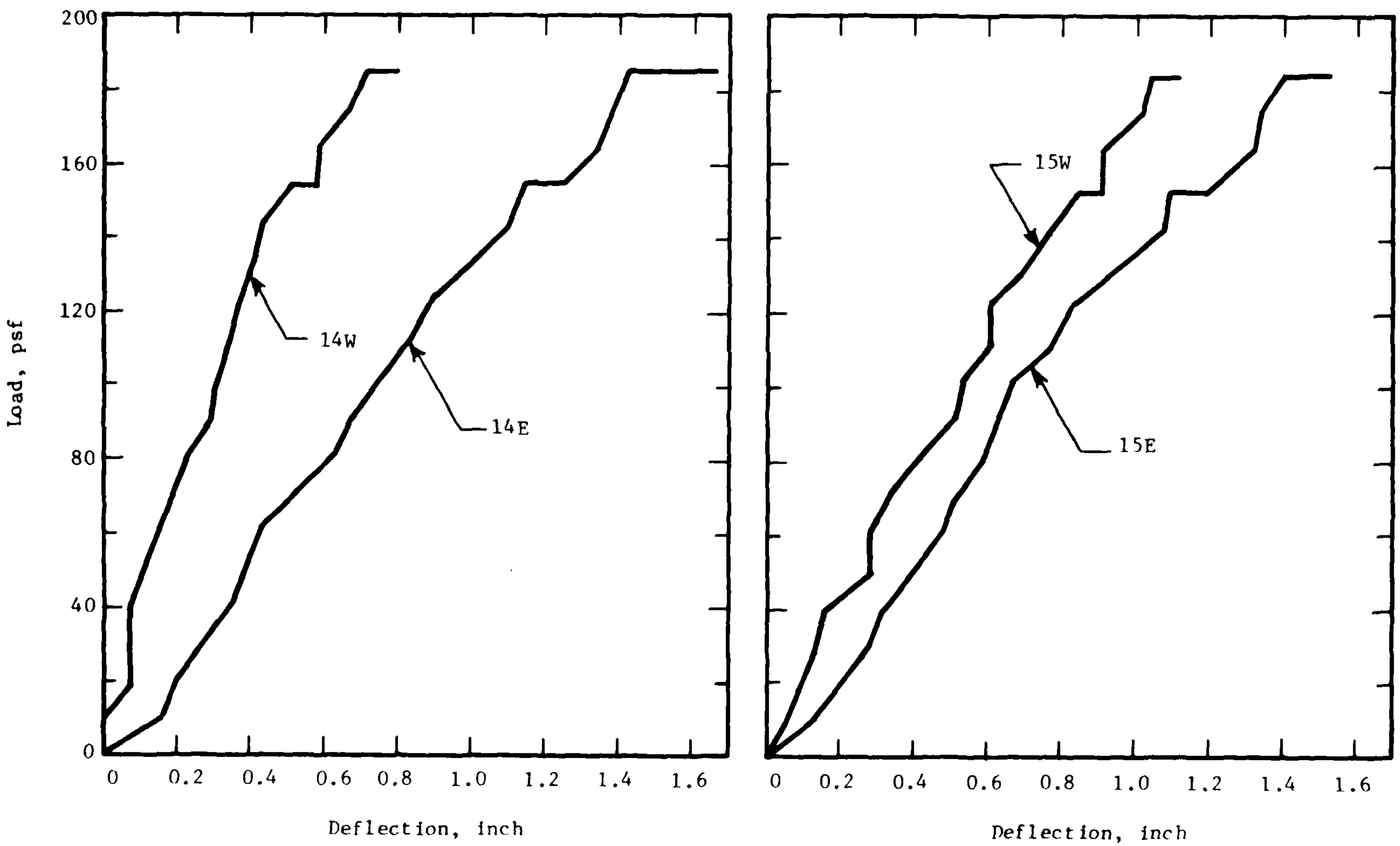
Figure A.3 Joist Deflections (continued)



(e) Joist 12

(f) Joist 13

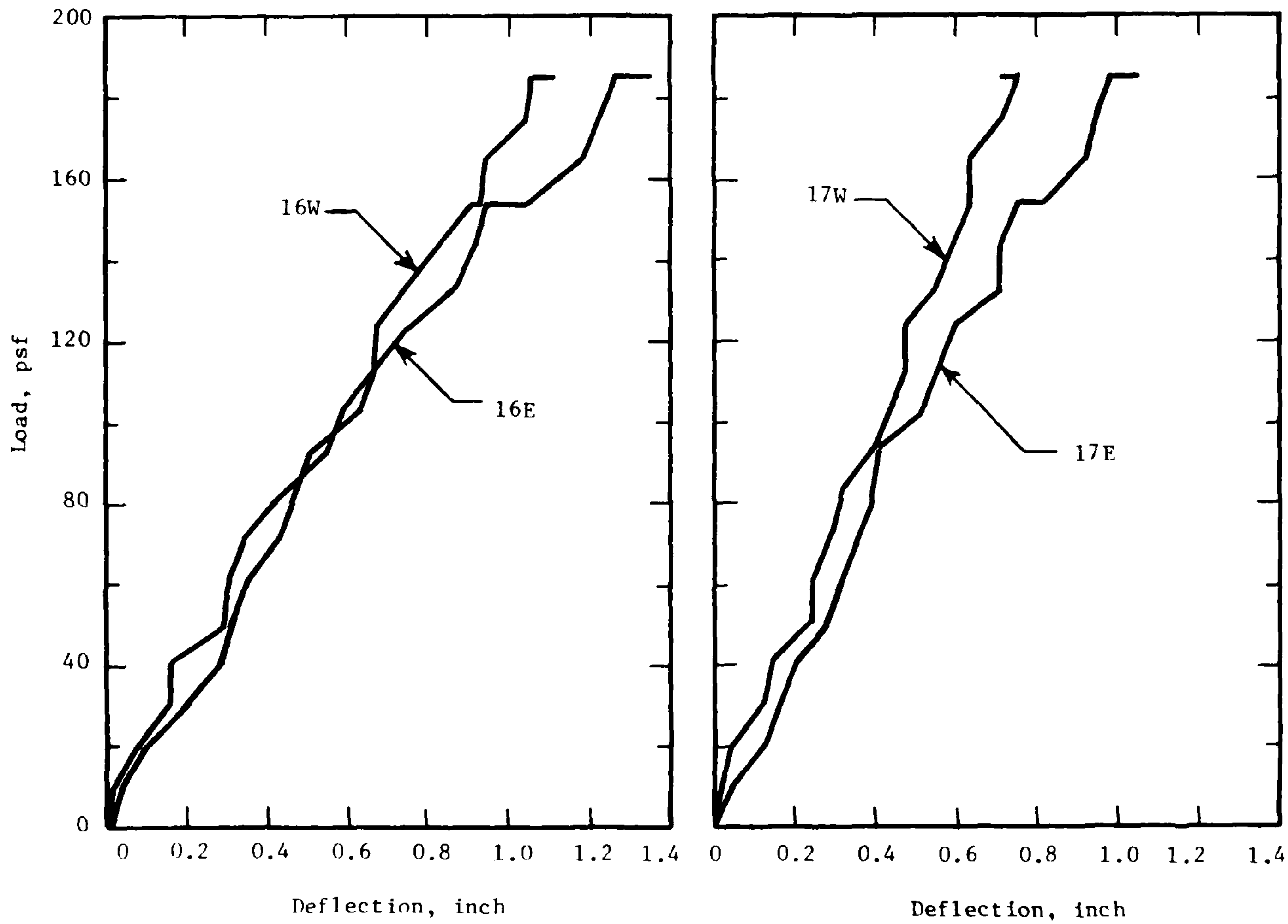
Figure A.3 Joist Deflections (continued)



(g) Joist 14

(h) Joist 15

Figure A.3 Joist Deflections (continued)



(i) Joist 16

(j) Joist 17

Figure A.3 Joist Deflections (concluded)

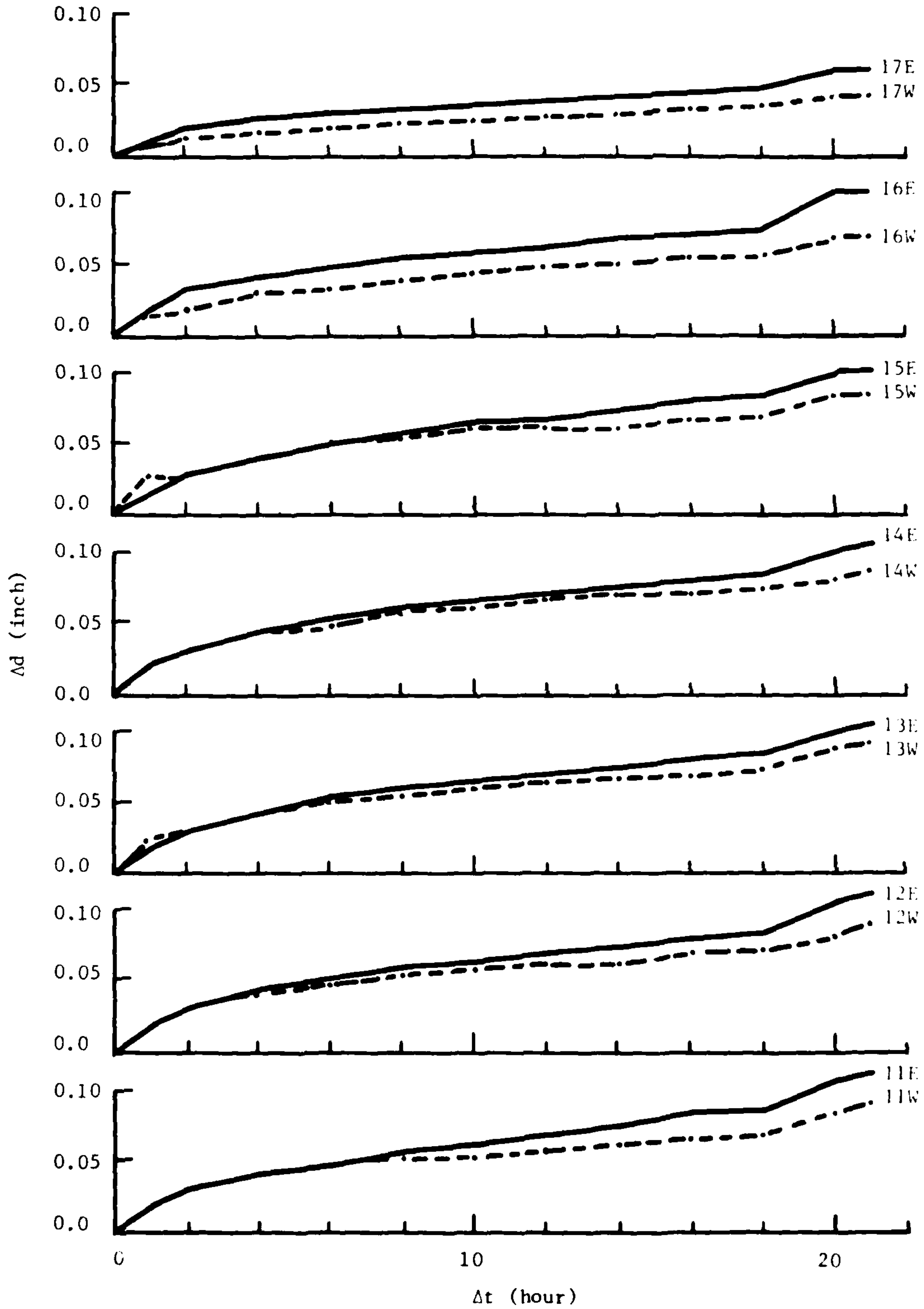


Figure A.4(a) Time Deflection at Constant Load (154.5 psf) for Joists, First Load Test

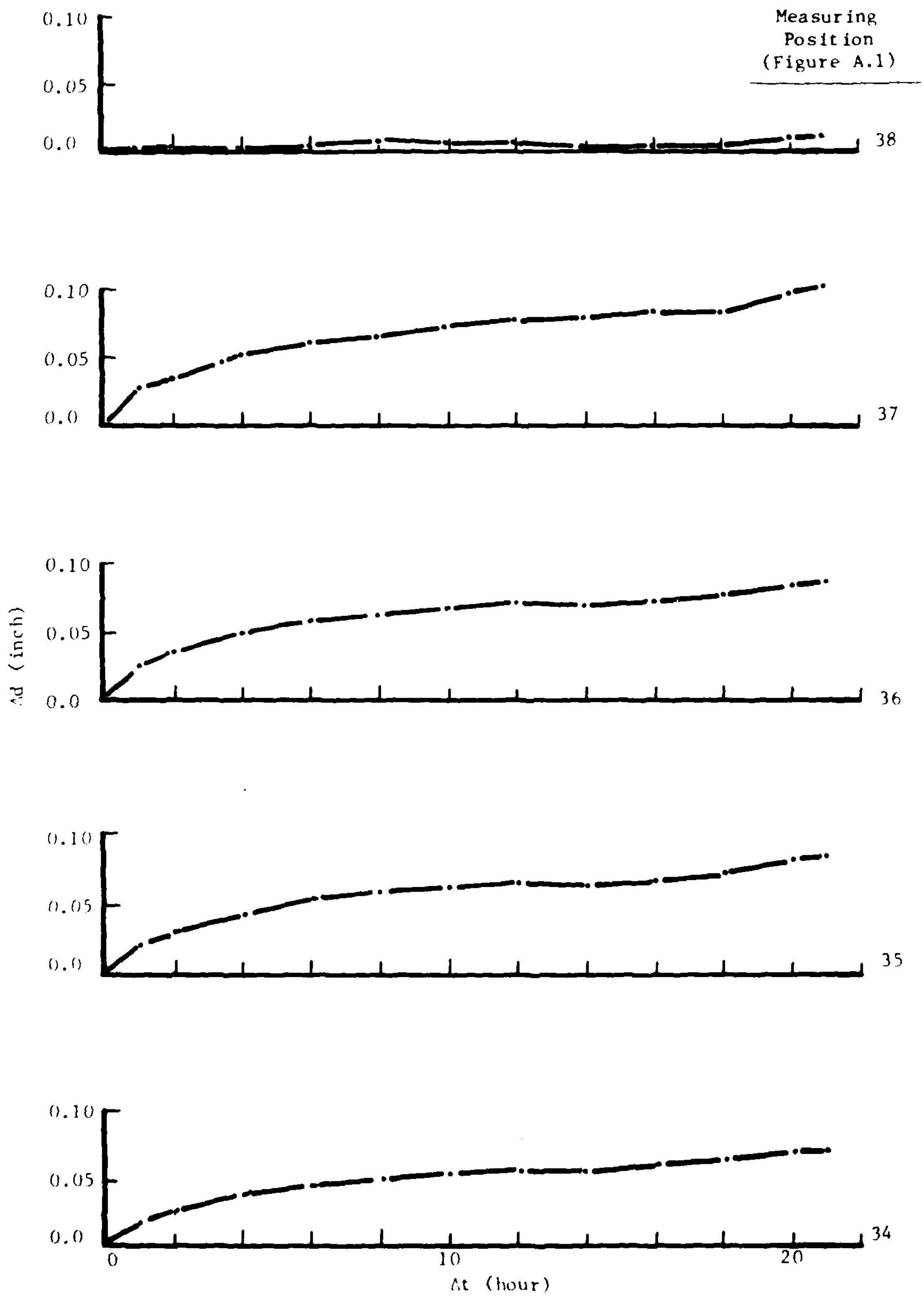


Figure A.4(b) Time Deflection at Constant load (154.5 psf) for Girder, First Load Test

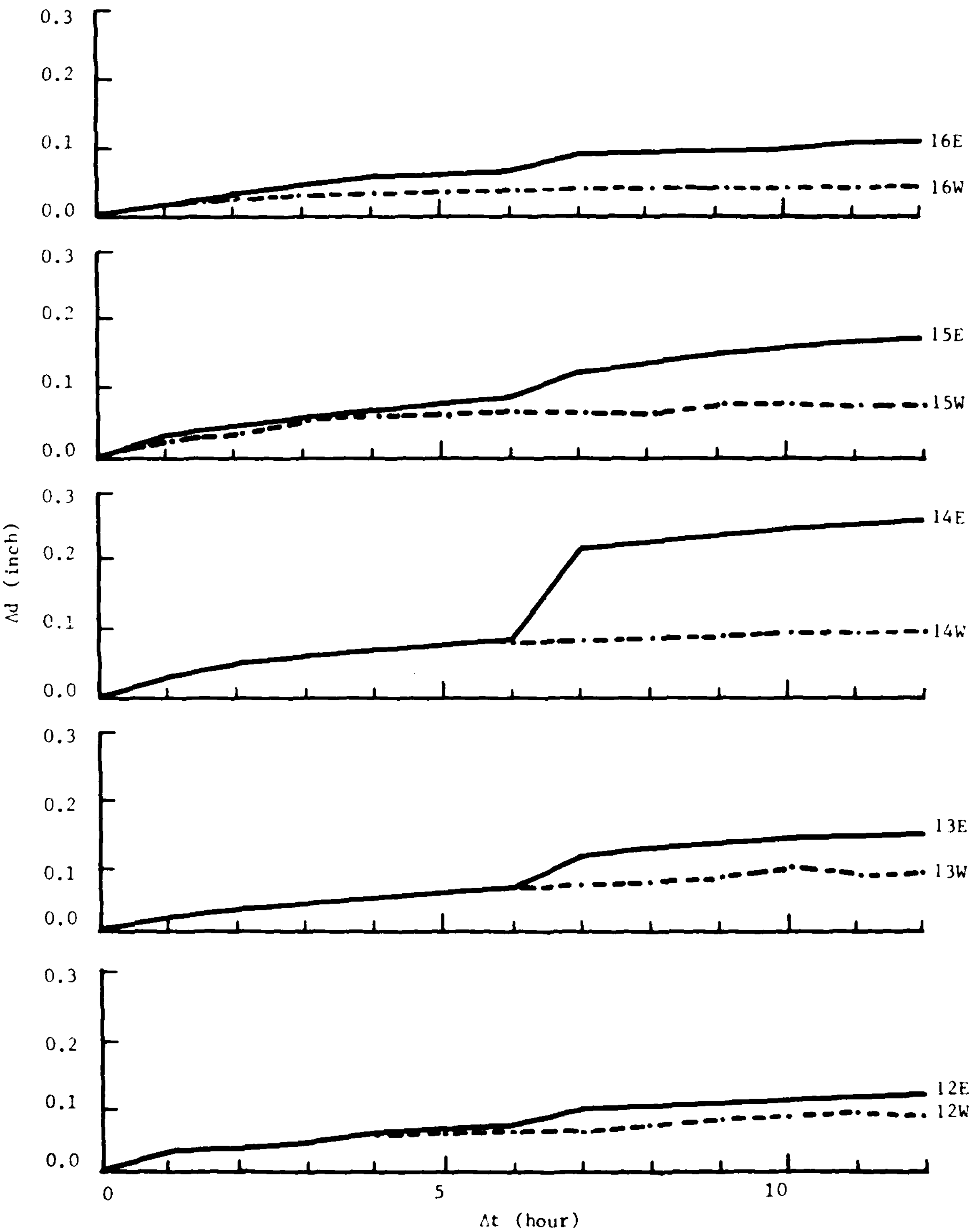


Figure A.5(a) Time Deflection at Constant Load (185.4 psf) for Joists, First Load Test

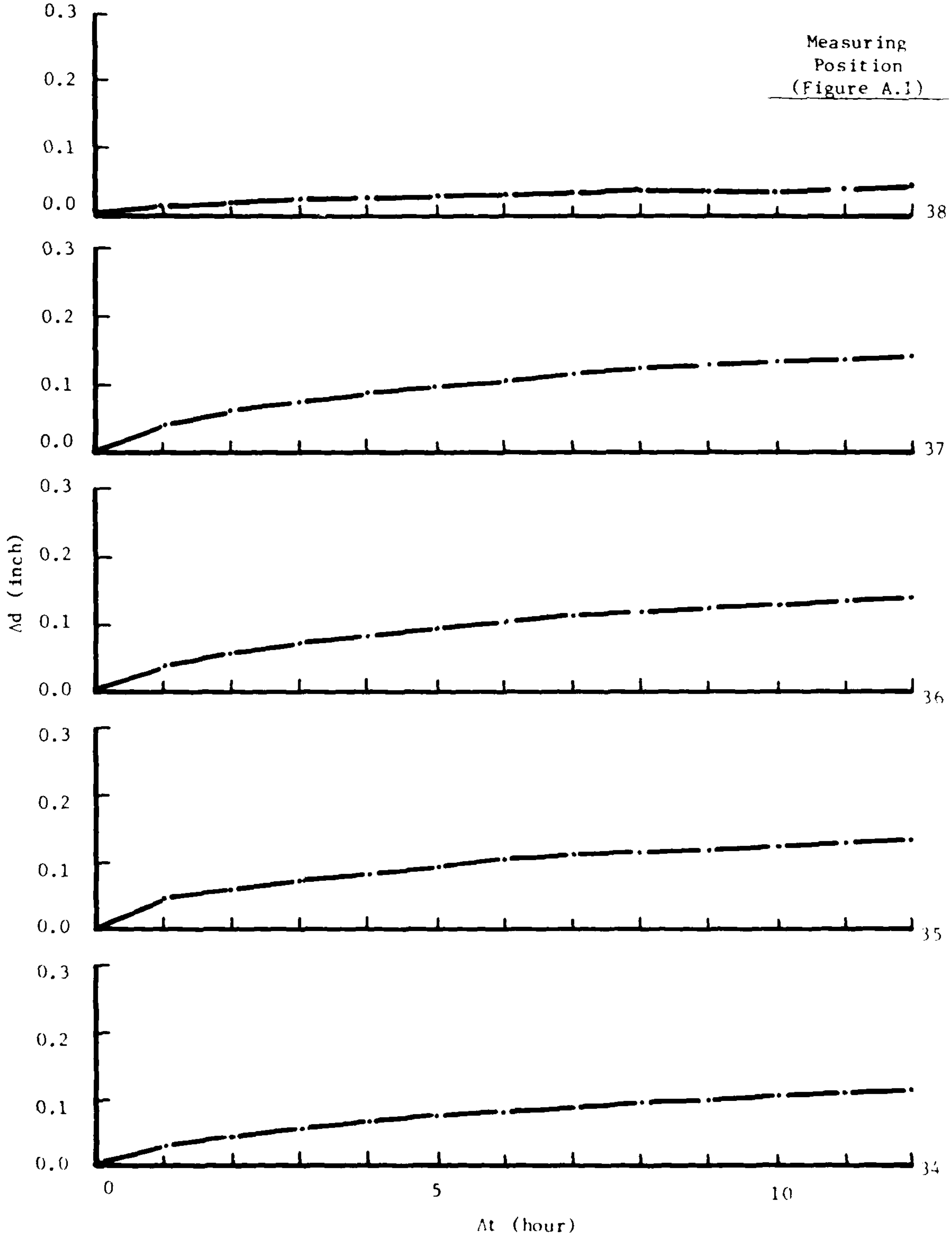


Figure A.5(b) Time Deflection at Constant Load (185.4 psf) for Girder, First Load Test

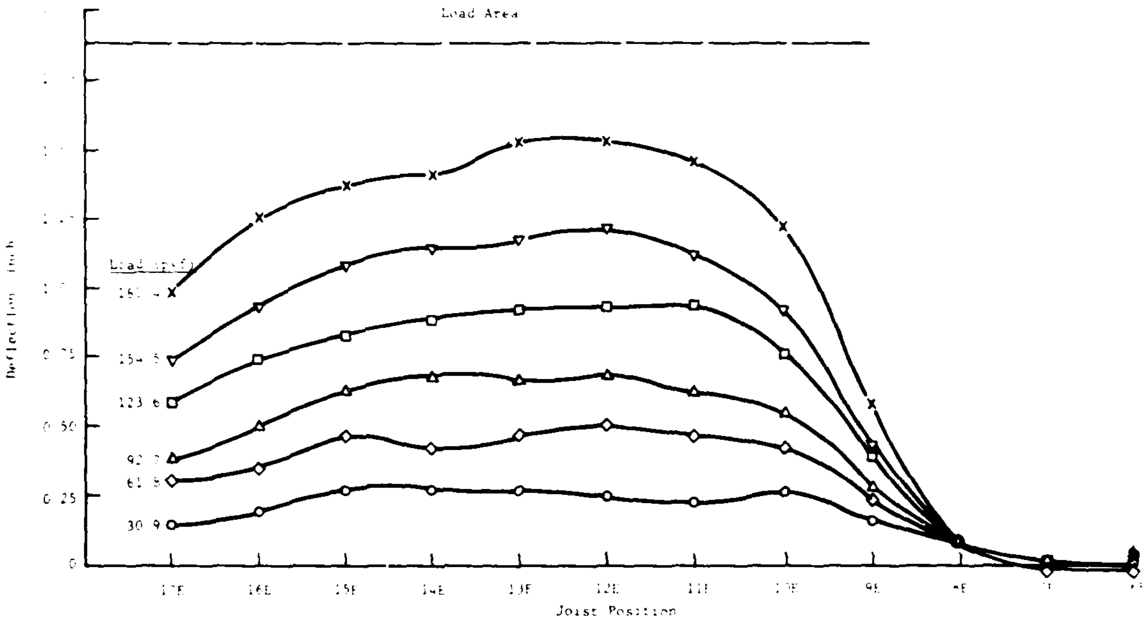


Figure A.6 Deflection versus Joist Position for Indicated Load Levels - East Side, First Load Test

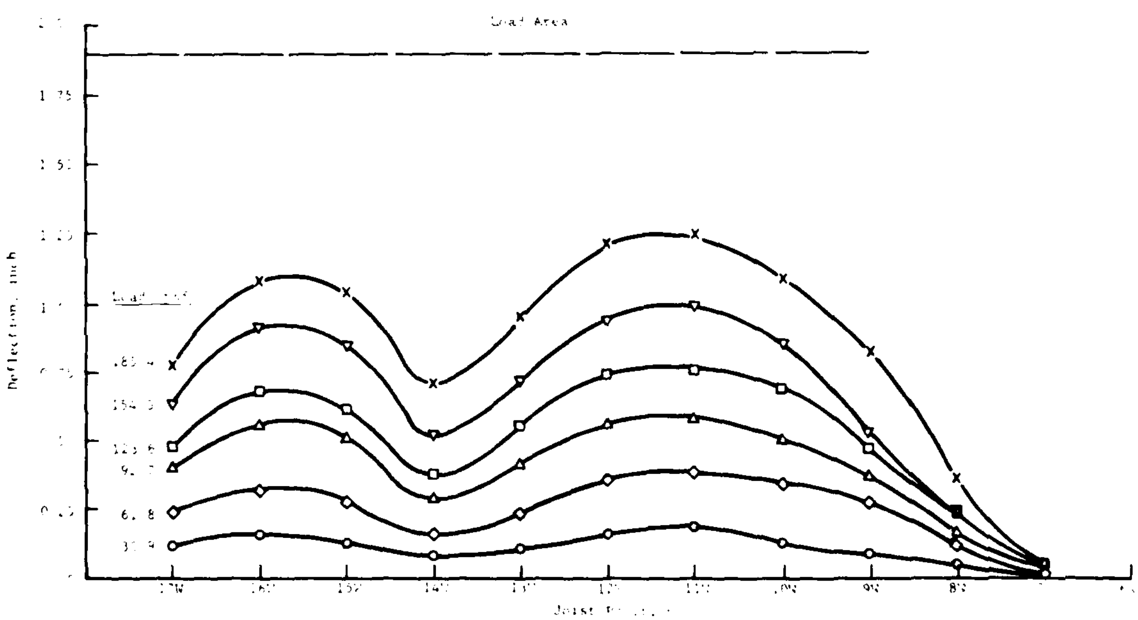


Figure A.7 Deflection versus Joist Position for Indicated Load Levels - West Side, First Load Test

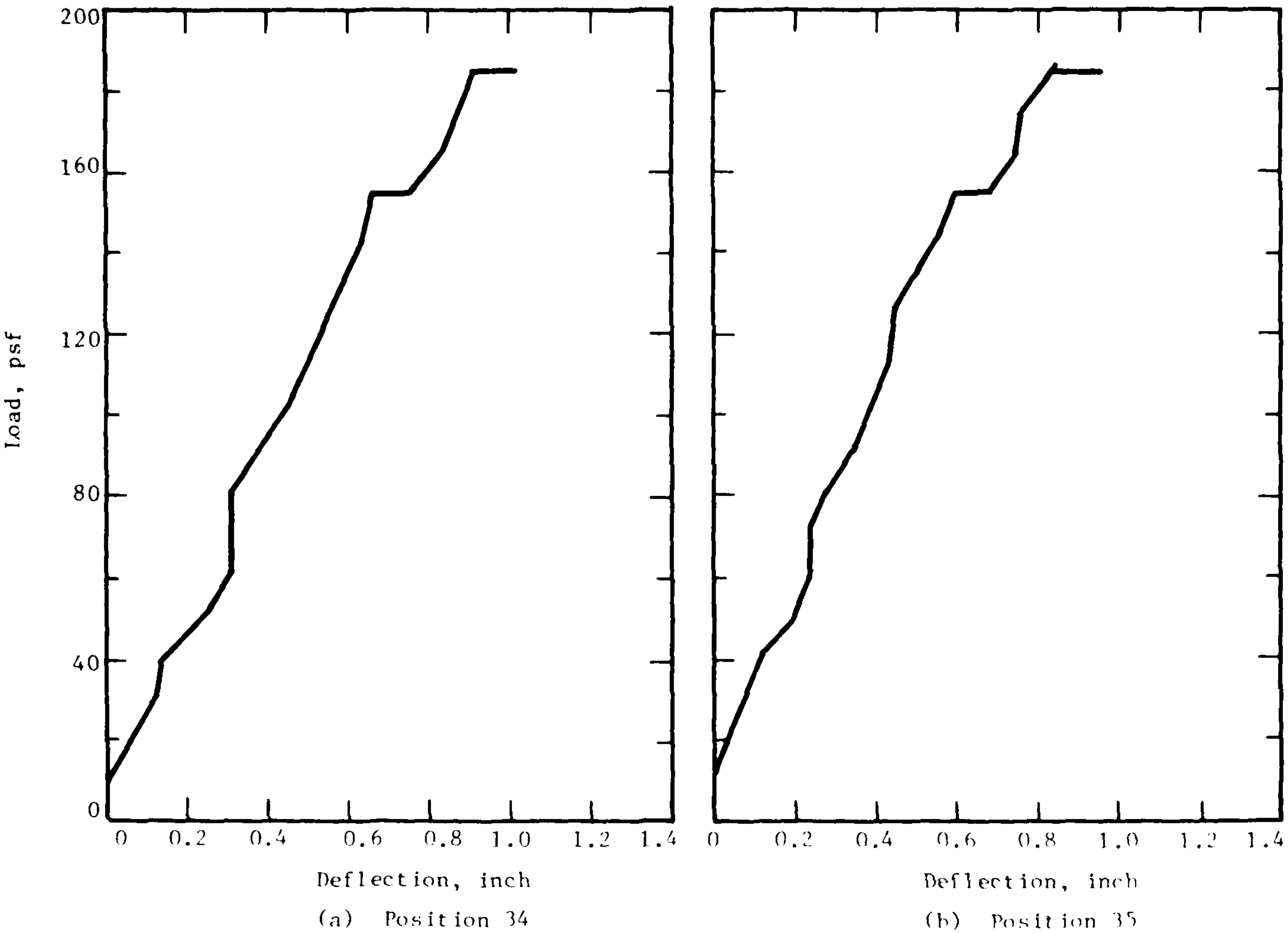
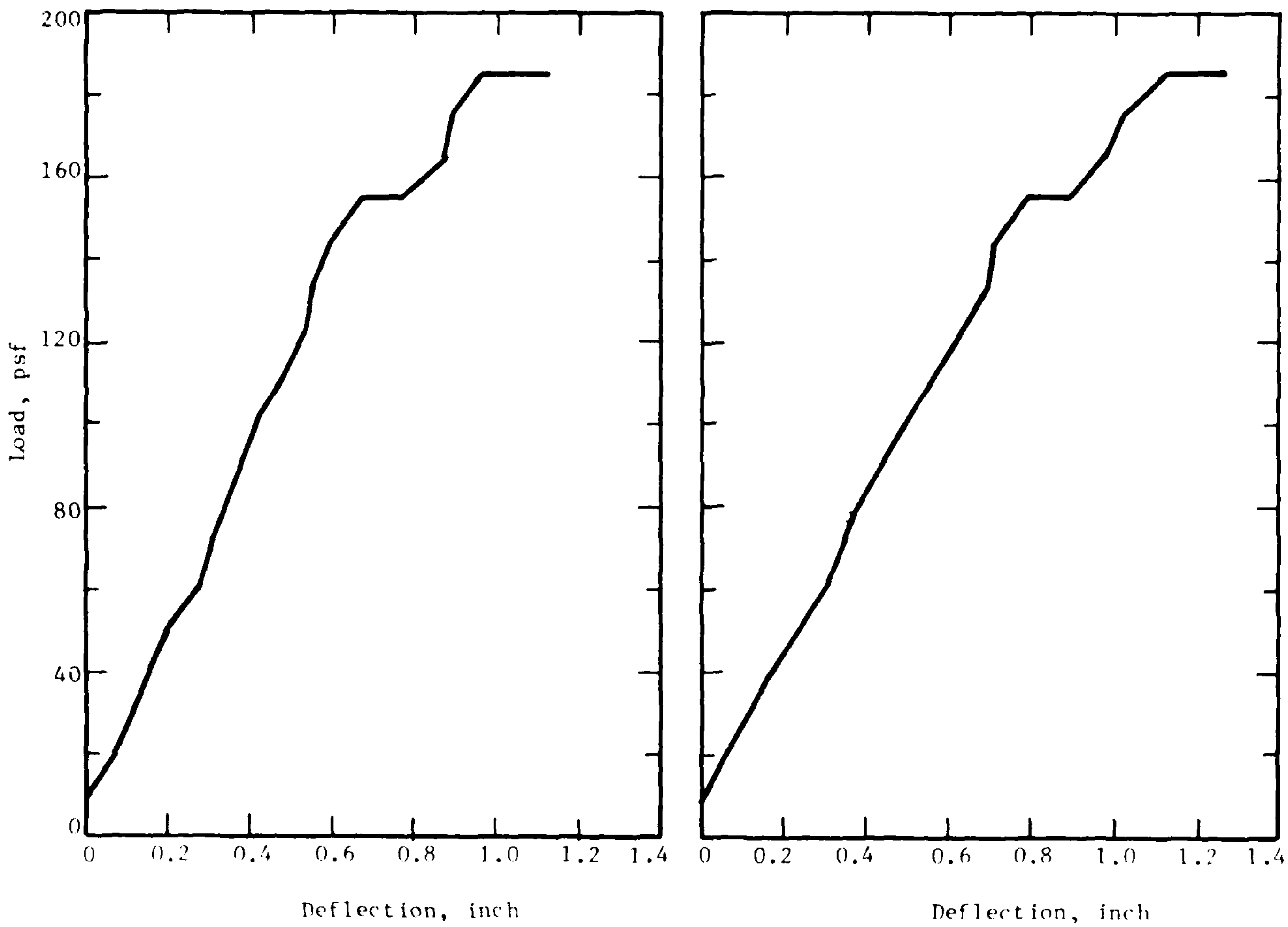


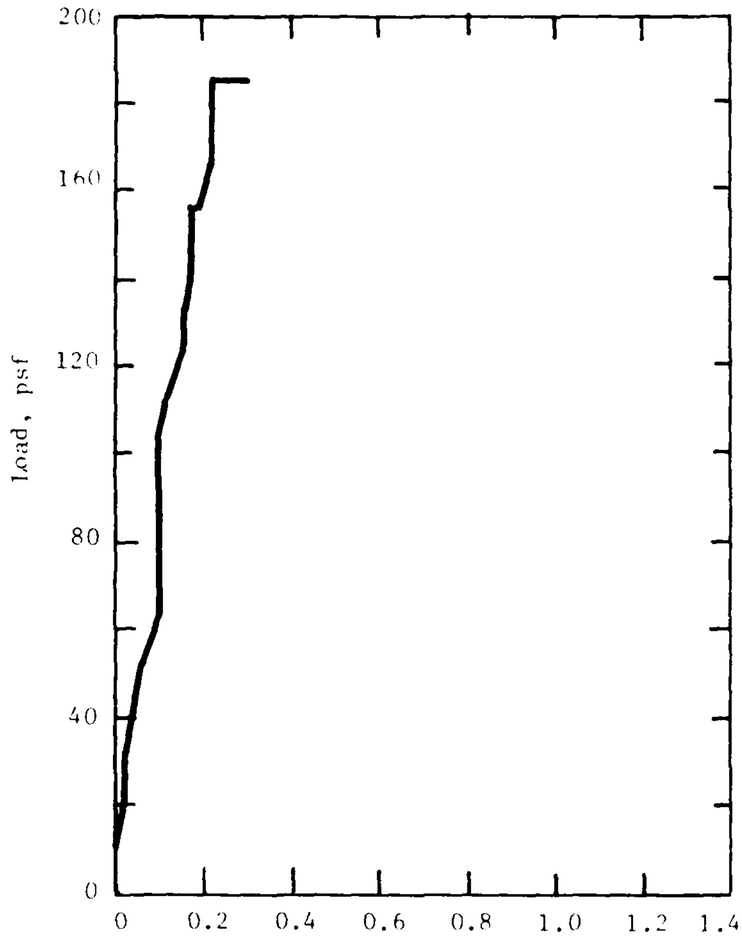
Figure A.8 Girder Deflections



(c) Position 36

(d) Position 37

Figure A.8 Girder Deflections (continued)



(e) Position 38

Figure A.8 Girder Deflections (concluded)

A.2 Second Load Test

For this load test, the deflection measurement instrumentation layout is shown in Figure A.9. As in the previous test, this included joist and girder deflections. Note, measuring positions 34, 36 and 38 are at the respective midspans of the girder. Measuring positions 35 and 37 are at one-half midspan.

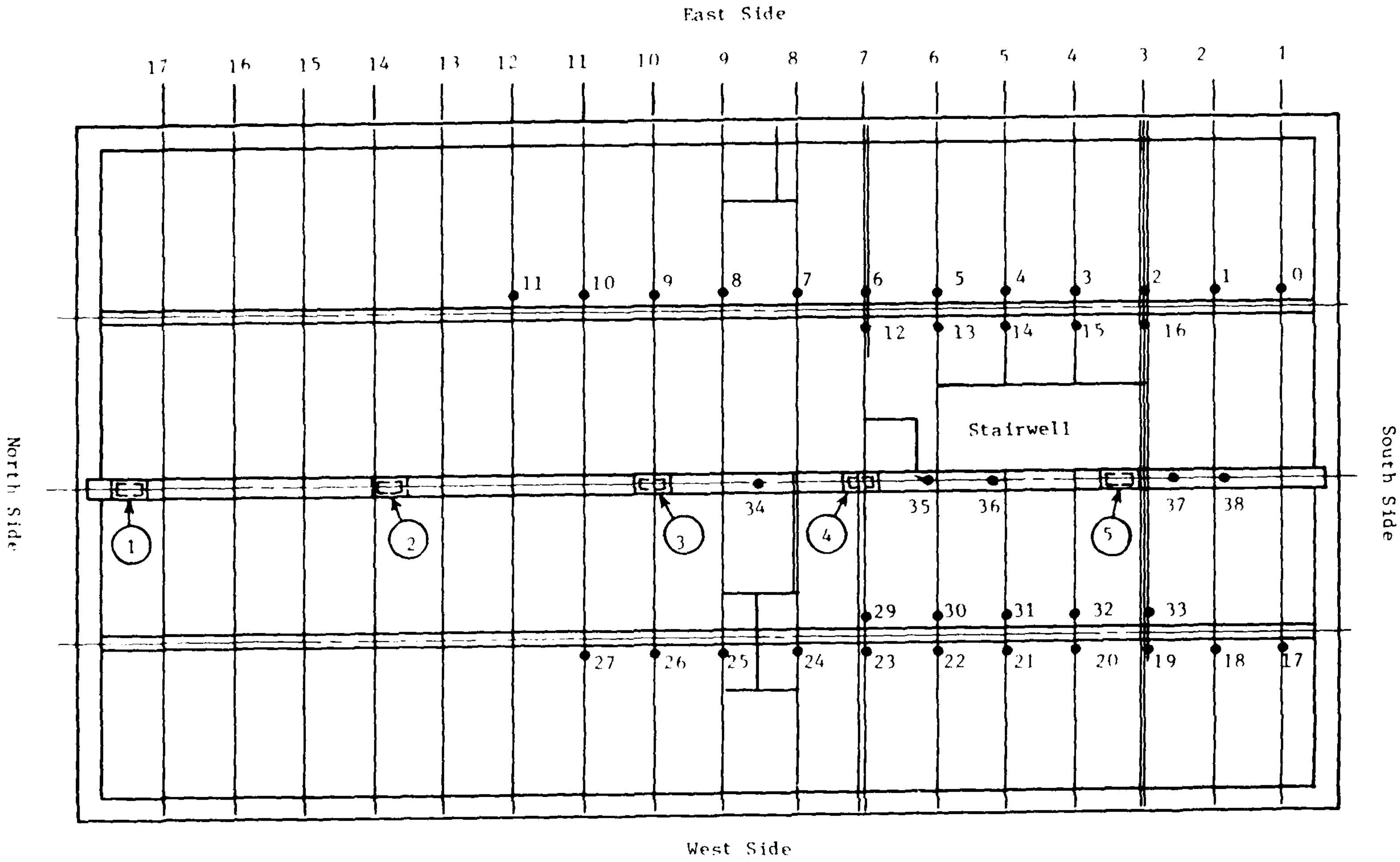


Figure A.9 Deflection Measurement Instrumentation Layout, Second Load Test

The loading sequence for this test is shown schematically in Figure A.10. The test structure was loaded up to 11 pallets (130.9 psf) in equal increments during the time of 3.7 hr. This load was maintained constant for approximately 50 minutes. The load was then increased in equal increments up to 39 pallets (464.1 psf) and maintained constant for 12 hr. The load was subsequently increased to 47 pallets (559.30 psf) and left on for about 1 hr. The structure was unloaded in approximately 4 hr.

Average joist deflections for joists 3 to 7 are shown in Figure A.11. Time deflections at the constant load of 464.1 psf during the 12 hr time period are shown in Figure A.12 for joists 3 through 7.

Joist deflections as a function of position are given in Figures A.13 and A.14 for the east and west sides of the test structure. With the exception of moderate crushing at points of support, this test structure experienced no failure.

Detailed deflection data for the second load test are given in Tables A.1 and A.2. Table A.1 contains joist deflection measurements as a function of load. The load is given in one pallet increments, one pallet is approximately 3600 lb. Table A.2 contains girder deflections as a function of load also in one pallet increments. Deflections are given in centimeters in both tables. Measuring positions (channels) are given in Figure A.9.

Note: 1 Pallet = 11.9 psf

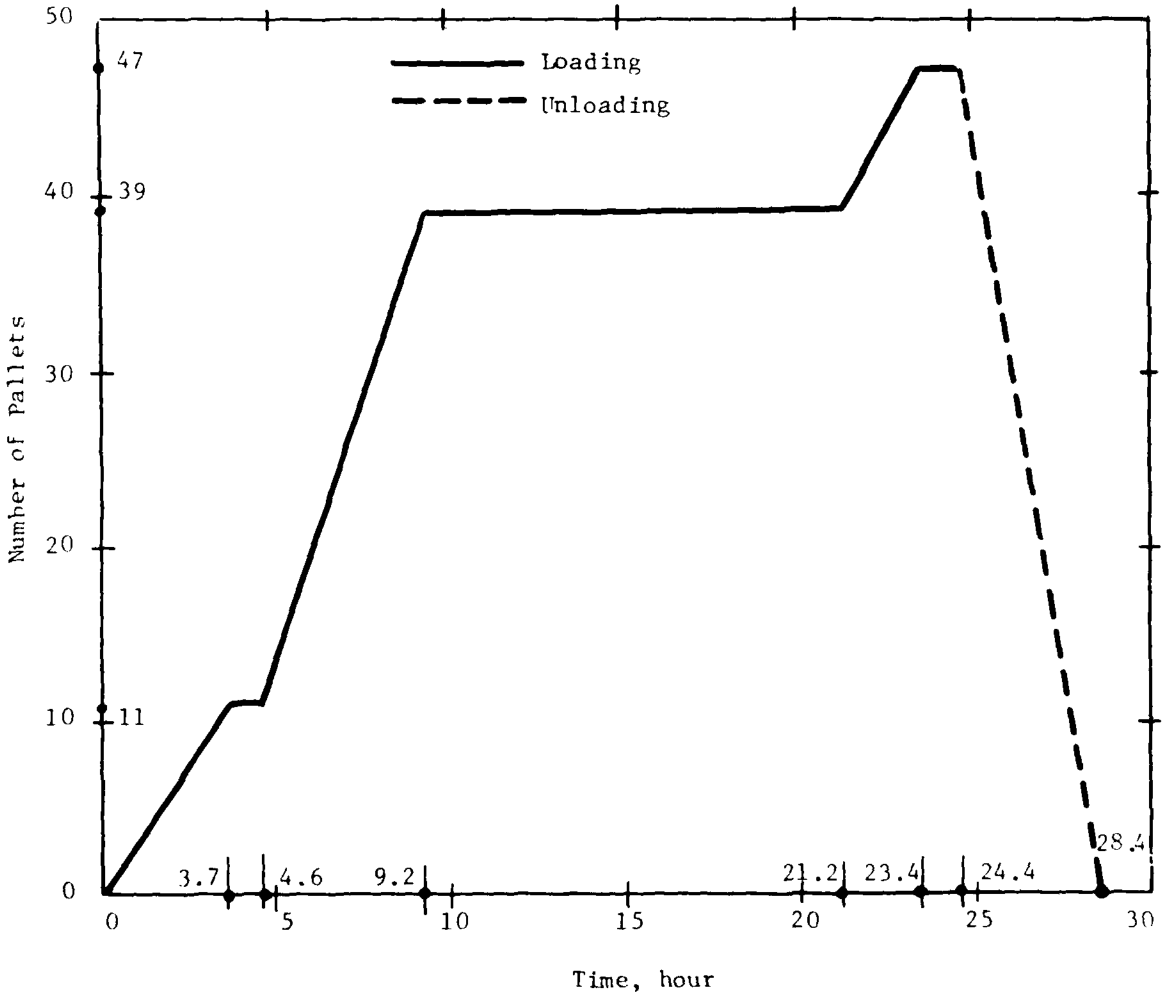


Figure A.10 Load-Time Diagram for the Second Load Test

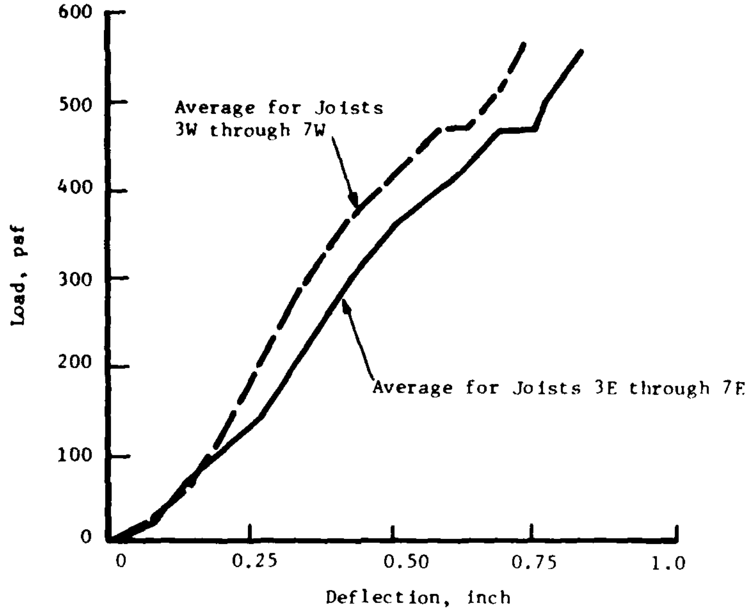


Figure A.11 Average Load-Deflection Curves for Joists, Second Load Test

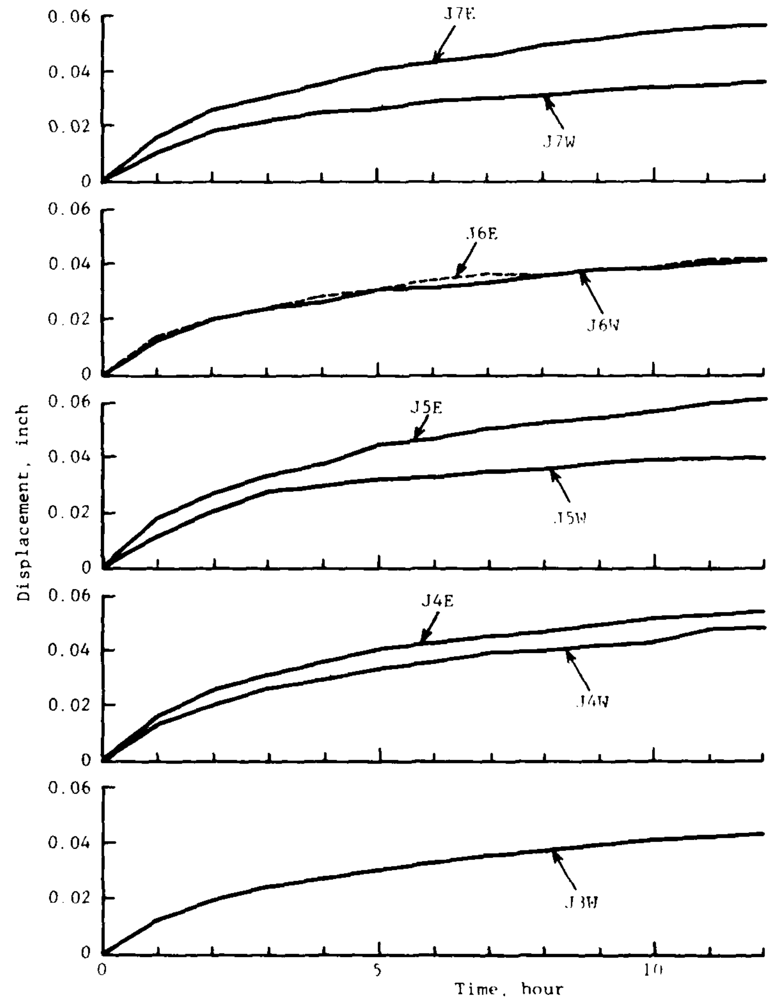


Figure A.12 Time Displacement at a Constant Load - 464.1 psf

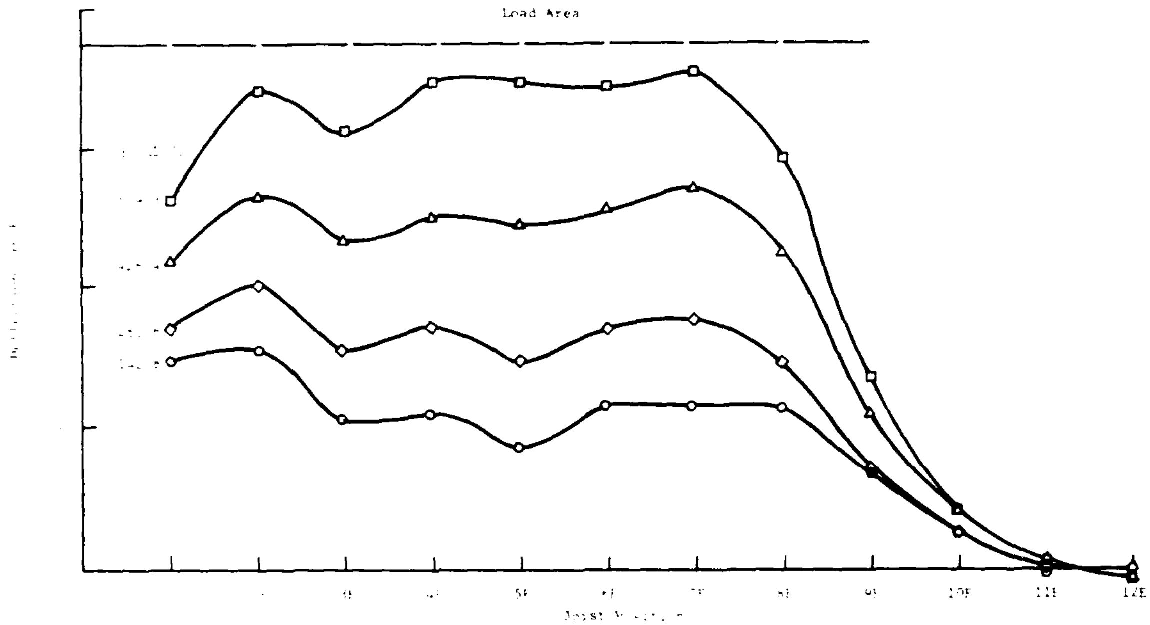


Figure A.13 Deflection versus Joist Position for Indicated Load Levels - East Side, Second Load Test

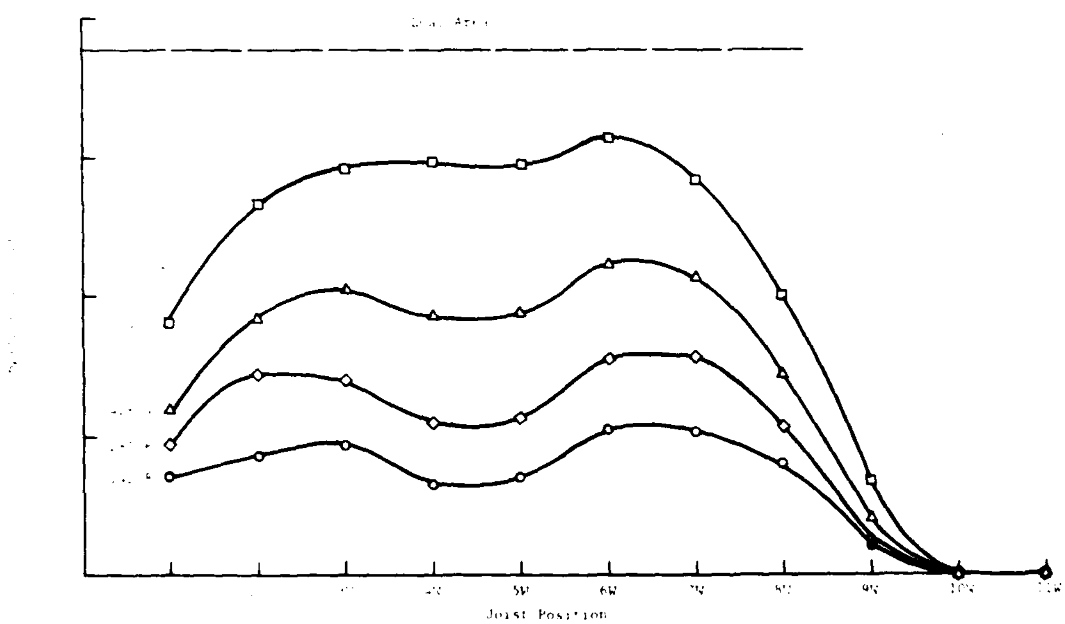


Figure A.14 Deflection versus Joist Position for Indicated Load Levels - West Side, Second Load Test

TABLE A.1

SECOND LOAD TEST JOIST DEFLECTION MEASUREMENTS

	Joist/Channel* Deflection (cm)											
Pallet	1E/0	2E/1	3E/2	4E/3	5E/4	6E/5	7E/6	8E/7	9E/8	10E/9	11E/10	12E/11
0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0.05	0.1	0.15	0.10	0.20	0.15	0.25	0.20	0.10	0	0
2	0.2	0.25	0.25	0.20	0.15	0.20	0.20	0.35	0.22	0.10	0	0
3	0.4	0.45	0.40	0.25	0.17	0.35	0.30	0.40	0.20	0.10	0	0.03
4	0.4	0.40	0.40	0.35	0.25	0.40	0.35	0.40	0.23	0.10	0	0
5	0.55	0.60	0.40	0.35	0.25	0.40	0.35	0.42	0.24	0.05	-0.02	-0.05
6	0.60	0.60	0.40	0.38	0.25	0.45	0.45	0.50	0.25	0.09	-0.02	-0.03
7	0.65	0.70	0.40	0.40	0.30	0.50	0.45	0.53	0.30	0.08	-0.02	-0.03
8	0.70	0.70	0.40	0.40	0.33	0.55	0.50	0.55	0.29	0.08	-0.03	-0.05
9	0.80	0.75	0.50	0.48	0.35	0.57	0.55	0.55	0.29	0.10	-0.01	-0.03
10	0.90	0.90	0.60	0.60	0.45	0.64	0.65	0.63	0.33	0.12	0.01	-0.03
11	0.92	0.98	0.62	0.68	0.53	0.74	0.67	0.65	0.35	0.15	0.01	0
12	0.95	1.00	0.68	0.70	0.55	0.75	0.75	0.73	0.43	0.17	0.02	0
13	0.95	1.05	0.70	0.80	0.60	0.80	0.77	0.73	0.43	0.18	0.04	0
14	1.0	1.10	0.72	0.80	0.60	0.83	0.79	0.73	0.40	0.17	0.02	0
15	1.08	1.15	0.78	0.82	0.65	0.85	0.83	0.76	0.41	0.18	0.02	0
16	1.05	1.17	0.80	0.88	0.72	0.93	0.87	0.77	0.41	0.17	0.02	0
17	1.08	1.18	0.80	0.88	0.73	0.93	0.89	0.77	0.43	0.17	0.02	-0.01
18	1.05	1.20	0.82	0.91	0.75	0.95	0.93	0.80	0.44	0.18	0.02	-0.02
19	1.10	1.20	0.88	0.94	0.76	0.96	0.95	0.83	0.45	0.18	0.02	-0.02
20	1.10	1.20	0.88	0.94	0.81	0.99	0.96	0.83	0.44	0.17	0.01	-0.03
21	1.10	1.20	0.90	1.00	0.85	1.03	1.01	0.85	0.45	0.18	0.01	-0.03
22	1.10	1.20	0.90	1.00	0.85	1.05	1.05	0.86	0.45	0.18	0	-0.03
23	1.10	1.25	0.95	1.09	0.93	1.02	1.09	0.89	0.44	0.17	0	-0.03
24	1.10	1.30	1.00	1.10	0.97	1.10	1.14	0.94	0.46	0.16	0	-0.03
25	1.15	1.30	1.01	1.15	1.00	1.14	1.21	1.00	0.53	0.19	0.02	-0.03
26	1.15	1.30	1.05	1.17	1.05	1.20	1.25	1.03	0.53	0.19	0.02	-0.03
27	1.20	1.38	1.08	1.20	1.10	1.23	1.28	1.03	0.53	0.20	0.02	-0.03
28	1.20	1.40	1.15	1.24	1.15	1.29	1.34	1.06	0.54	0.20	0.02	-0.03
29	1.20	1.40	1.17	1.30	1.22	1.33	1.37	1.10	0.55	0.21	0.01	-0.03
30	1.20	1.48	1.20	1.35	1.25	1.35	1.40	1.13	0.56	0.22	0.02	-0.03
31	1.25	1.50	1.25	1.40	1.27	1.39	1.46	1.18	0.59	0.22	0.02	-0.03
32	1.30	1.55	1.30	1.42	1.37	1.45	1.53	1.23	0.63	0.22	0.02	-0.03
33	1.35	1.60	1.35	1.50	1.45	1.47	1.56	1.25	0.63	0.24	0.02	-0.03
34	1.37	1.60	1.38	1.50	1.45	1.50	1.60	1.27	0.63	0.22	0.01	-0.03
35	1.40	1.62	1.40	1.50	1.50	1.55	1.65	1.34	0.65	0.25	0.01	-0.03

TABLE A.1 (continued)

SECOND LOAD TEST JOIST DEFLECTION MEASUREMENTS

Pallet	Joist/Channel* Deflection (cm)											
	1E/0	2E/1	3E/2	4E/3	5E/4	6E/5	7E/6	8E/7	9E/8	10E/9	11E/10	12E/11
36	1.40	1.70	1.50	1.60	1.57	1.64	1.74	1.44	0.70	0.28	0.04	-0.02
37	1.42	1.70	1.50	1.68	1.64	1.66	1.79	1.46	0.73	0.28	0.03	-0.02
38	1.50	1.80	1.58	1.68	1.68	1.72	1.85	1.52	0.75	0.28	0.03	-0.02
39	1.50	1.88	1.65	1.78	1.77	1.75	1.87	1.53	0.75	0.28	0.02	-0.03
39	1.60	2.0	1.80	1.95	1.95	1.87	2.04	1.70	0.83	0.28	0.02	-0.03
40	1.60	2.0	1.80	1.99	1.97	1.90	2.07	1.73	0.82	0.28	0.02	-0.03
41	1.60	1.98	1.82	2.00	1.97	1.93	2.07	1.73	0.83	0.28	0.02	-0.03
42	1.60	2.00	1.82	2.00	2.03	1.95	2.11	1.73	0.84	0.28	0.02	-0.03
43	1.60	2.05	1.85	2.02	2.05	1.95	2.13	1.75	0.85	0.29	0.02	-0.03
44	1.62	2.10	1.90	2.08	2.09	2.00	2.16	1.77	0.84	0.28	0.02	-0.03
45	1.70	2.10	1.90	2.10	2.13	2.03	2.23	1.80	0.87	0.28	0.02	-0.03
46	1.70	2.15	2.00	2.15	2.17	2.04	2.25	1.85	0.87	0.28	0.02	-0.03
47	1.68	2.18	2.00	2.20	2.23	2.05	2.27	1.87	0.87	0.28	0.01	-0.04
0	0.45	0.60	0.62	0.70	0.65	0.63	1.20	0.53	0.25	0.18	0.02	-0.02
	1W/17	2W/18	3W/19	4W/20	5W/21	6W/22	7W/23	8W/24	9W/25	10W/26	11W/27	
0	0	0	0	0	0	0	0	0	0	0	0	
1	-0.05	0	0.05	0.05	0.20	0.20	0.10	0.13	-0.02	-0.05	-0.02	
2	0.10	0.15	0.18	0.13	0.15	0.25	0.14	0.19	0.04	-0.02	-0.01	
3	0.15	0.20	0.27	0.20	0.20	0.30	0.20	0.25	0.05	0	0	
4	0.15	0.20	0.27	0.12	0.21	0.35	0.28	0.28	0.04	-0.04	-0.05	
5	0.20	0.31	0.40	0.23	0.29	0.44	0.33	0.33	0.05	-0.03	-0.04	
6	0.25	0.31	0.40	0.28	0.29	0.47	0.40	0.35	0.05	-0.03	-0.02	
7	0.30	0.39	0.46	0.30	0.35	0.54	0.45	0.40	0.07	-0.01	0	
8	0.34	0.41	0.46	0.31	0.33	0.56	0.55	0.42	0.07	-0.02	-0.04	
9	0.30	0.44	0.50	0.31	0.38	0.64	0.57	0.43	0.09	-0.01	-0.02	
10	0.35	0.49	0.54	0.35	0.39	0.64	0.62	0.47	0.10	-0.01	-0.01	
11	0.37	0.51	0.60	0.40	0.41	0.64	0.57	0.48	0.12	-0.02	-0.02	
12	0.45	0.55	0.60	0.42	0.45	0.66	0.66	0.51	0.13	0.01	0.01	
13	0.45	0.60	0.66	0.49	0.45	0.70	0.70	0.52	0.13	0	0.02	
14	0.45	0.60	0.66	0.47	0.45	0.73	0.74	0.57	0.14	0	0.02	
15	0.50	0.69	0.70	0.51	0.51	0.76	0.75	0.54	0.14	0	0.02	

TABLE A.1 (concluded)

SECOND LOAD TEST JOIST DEFLECTION MEASUREMENTS

	Joist/Channel* Deflection (cm)										
Pallet	1W/17	2W/18	3W/19	4W/20	5W/21	6W/22	7W/23	8W/24	9W/25	10W/26	11W/27
16	0.50	0.70	0.75	0.52	0.52	0.78	0.76	0.56	0.14	0	0.02
17	0.52	0.71	0.75	0.54	0.56	0.80	0.78	0.58	0.14	0	-0.04
18	0.56	0.75	0.80	0.60	0.59	0.83	0.82	0.61	0.16	0.03	0.01
19	0.54	0.81	0.85	0.62	0.61	0.84	0.85	0.61	0.16	0.01	0
20	0.57	0.87	0.86	0.65	0.61	0.87	0.87	0.64	0.16	0	-0.04
21	0.60	0.90	0.88	0.70	0.70	0.94	0.95	0.67	0.17	0.01	-0.03
22	0.65	0.90	0.94	0.72	0.72	0.97	0.96	0.68	0.17	0.01	-0.02
23	0.66	0.92	0.95	0.74	0.79	1.04	1.01	0.70	0.19	0.02	-0.02
24	0.61	0.92	0.90	0.70	0.72	0.99	1.00	0.68	0.16	-0.01	-0.05
25	0.65	0.95	0.95	0.72	0.76	1.03	1.05	0.72	0.21	0	-0.04
26	0.65	0.95	0.97	0.79	0.80	1.06	1.09	0.72	0.21	0	-0.04
27	0.66	1.00	1.02	0.82	0.85	1.09	1.10	0.74	0.23	+0.03	-0.03
28	0.67	1.01	1.05	0.90	0.91	1.15	1.13	0.76	0.23	0.01	-0.04
29	0.69	1.01	1.06	0.91	0.92	1.15	1.16	0.79	0.24	0.01	-0.04
30	0.72	1.02	1.11	0.94	0.99	1.21	1.21	0.80	0.25	0.02	-0.02
31	0.74	1.09	1.15	1.00	1.03	1.26	1.24	0.83	0.25	+0.01	-0.04
32	0.76	1.11	1.21	1.01	1.04	1.27	1.26	0.86	0.26	0.03	-0.04
33	0.76	1.12	1.16	1.06	1.05	1.27	1.26	0.88	0.26	-0.01	-0.05
34	0.76	1.15	1.21	1.10	1.10	1.35	1.35	0.90	0.26	0.01	-0.05
35	0.76	1.11	1.25	1.10	1.12	1.37	1.35	0.90	0.25	0	-0.06
36	0.76	1.18	1.31	1.18	1.20	1.43	1.37	0.92	0.26	0.02	-0.06
37	0.77	1.21	1.35	1.22	1.21	1.45	1.40	0.92	0.25	0.01	-0.06
38	0.90	1.31	1.45	1.32	1.38	1.56	1.49	1.01	0.30	0.04	-0.05
39	0.95	1.40	1.55	1.45	1.40	1.63	1.54	1.02	0.34	0.05	-0.02
39	0.95	1.50	1.66	1.58	1,58	1.76	1.65	1.12	0.36	0.04	-0.05
40	1.00	1.53	1.75	1.61	1.62	1.77	1.67	1.15	0.37	0.05	-0.04
41	1.03	1.58	1.75	1.69	1.67	1.85	1.69	1.16	0.35	0.05	-0.04
42	1.03	1.54	1.79	1.72	1.72	1.86	1.71	1.19	0.37	0.05	-0.03
43	1.07	1.60	1.80	1.72	1.72	1.88	1.73	1.20	0.39	0.05	-0.04
44	1.09	1.61	1.84	1.72	1.73	1.90	1.73	1.20	0.39	0.05	-0.03
45	1.10	1.66	1.86	1.80	1.79	1.93	1.75	1.22	0.41	0.05	-0.03
46	1.10	1.65	1.94	1.82	1.81	1.97	1.76	1.26	0.41	0.05	-0.05
47	1.15	1.70	1.36	1.89	1.88	2.03	1.81	1.28	0.43	0.05	-0.04
0	0.30	0.55	0.73	0.69	0.55	0.63	0.56	0.40	0.13	0.01	-0.01

*Channels (measuring positions) are shown in Figure A.9

TABLE A.2

SECOND TEST GIRDER DEFLECTION MEASUREMENTS

Pallet	Position and Channel Deflection (cm)				
	34	35	36	37	38
0	0	0	0	0	0
1	0.10	0.05	0	0	0
2	0.12	0.05	0.01	0.05	0.08
3	0.20	0.10	0.10	0.15	0.15
4	0.25	0.15	0.12	0.15	0.18
5	0.28	0.15	0.12	0.20	0.20
6	0.38	0.20	0.12	0.25	0.30
7	0.39	0.22	0.18	0.27	0.30
8	0.47	0.24	0.19	0.27	0.34
9	0.42	0.16	0.10	0.23	0.29
10	0.48	0.27	0.20	0.30	0.38
11	0.50	0.35	0.28	0.40	0.45
12	0.60	0.42	0.30	0.43	0.48
13	0.62	0.42	0.34	0.45	0.50
14	0.61	0.45	0.38	0.35	0.60
15	0.67	0.47	0.40	0.46	0.54
16	0.67	0.47	0.37	0.47	0.55
17	0.69	0.52	0.41	0.50	0.59
18	0.71	0.53	0.42	0.55	0.62
19	0.78	0.58	0.49	0.61	0.70
20	0.79	0.63	0.50	0.63	0.71
21	0.83	0.66	0.54	0.67	0.78
22	0.85	0.69	0.60	0.71	0.80
23	0.88	0.73	0.60	0.73	0.80
24	0.88	0.75	0.62	0.74	0.84
25	0.98	0.75	0.65	0.75	0.84
26	1.00	0.66	0.65	0.75	0.85
27	1.02	0.81	0.80	0.83	0.90
28	1.04	0.85	0.72	0.81	0.90
29	1.10	0.93	0.79	0.83	0.94
30	1.12	0.94	0.79	0.85	0.98
31	1.19	0.96	0.82	0.86	1.00
32	1.20	0.96	0.80	0.87	1.01
33	1.22	1.04	0.89	0.95	1.08
34	1.29	1.08	0.92	0.96	1.10
35	1.34	1.13	0.95	0.97	1.10
36	1.39	1.16	1.00	1.02	1.15
37	1.40	1.22	1.04	1.06	1.20
38	1.45	1.27	1.08	1.07	1.21
39	1.48	1.35	1.15	1.15	1.28
39/119	1.65	1.56	1.38	1.25	1.38
40	1.63	1.59	1.40	1.30	1.40
41	1.68	1.62	1.40	1.25	1.38
42	1.70	1.64	1.42	1.30	1.40
43	1.72	1.66	1.48	1.30	1.30
44	1.72	1.72	1.50	1.35	1.50
45	1.78	1.76	1.58	1.40	1.50
46	1.82	1.85	1.62	1.45	1.52
47	1.90	1.94	1.62	1.46	1.59
0	0.50	0.74	0.70	0.45	0.40

APPENDIX B

MECHANICAL PROPERTIES OF FLOOR JOISTS

A total of eighteen samples of wood were taken from the floor system after completion of the second load test. Eight were cut from joists and ten were cut from the flooring. All samples were identified by the Forest Products Laboratory (Ref 4) (Madison, Wisconsin) as Jack Pine.

Mean strength values for floor joists were determined for three time conditions of loading (see Table B.1) by adjusting the mean, clear wood strength values of Jack Pine obtained from the Wood Handbook (Ref 6). The size factor, was computed using the following expression (Ref 7).

$$\text{Size Factor} = \left(\frac{2}{d}\right)^{1/9} \quad \text{(B.1)}$$

where d = the depth of the joist

Strength and normal duration of load factors are from Tables 8 and 9 respectively of Ref 8.

Duration of load factors (ratios of allowable stress to that for normal loading) were obtained from Figure 10 of Ref 8. The three factors refer to the following load duration conditions and have the following values.

Load Duration Condition	Duration of Load Factor
(1) First load test. Average duration of maximum load was approximately 30 hr	1.32
(2) Laboratory tests. Average duration of maximum load for each test was approximately 1 hr	1.48
(3) Rapid application of load. For estimating strength under dynamic loading with load duration of about 1 sec	2.05

Strength values given in Table B.1 were rounded in accordance with Ref 7 as follows. F_b, F_c and F_t were rounded to the nearest 50 psi. F_v and $F_{c\perp}$ were rounded to the nearest 5 psi.

TABLE B.1

MECHANICAL PROPERTIES OF JOISTS*

Property**	Mean Clear Wood Strength Value, psi	Strength Factor***	Normal Duration of Load Factor	Size Factor	Combined Factor	Normal Duration Mean Strength Value, psi	1.25 day Duration Mean Strength Value, psi	1 hr Duration Mean Strength Value, psi	1 sec Duration Mean Strength Value, psi
F_b	9,900	0.63	1/1.6	0.89	0.35	3,450	4,600	5,150	7,100
F_c	5,660	0.78	2/3	----	0.52	2,950	3,900	4,350	6,050
F_v	1,170	0.50	1/1.6	----	0.31	365	480	715	750
F_t	9,900	0.37	1/1.6	----	0.23	2,300	3,000	4,450	4,700
$F_{c\perp}$	580	1.00	1/1.1	----	0.91	525	525	525	525
E/1000	1,350	1.00	1	----	1.00	1,350	1,350	1,350	1,350

*Jack Pine

**F_b = Rupture Strength

F_c = Compression Parallel to Grain

F_v = Shear Parallel to Grain

F_t = Tension Parallel to Grain

$F_{c\perp}$ = Compression Perpendicular to Grain

E = Modulus of Elasticity

***Structural Light Framing, Select Structural (Table 8, Ref D)

APPENDIX C

ANALYSIS OF TEST RESULTS

This appendix examines the test results with the object of understanding the behavior of the test floor system. Laboratory tests are considered first. This is then followed by results from the in situ load tests.

C.1 Laboratory Tests

Laboratory tests are described in Chapter 4. In this section we examine the results with the following objectives in mind.

(1) To determine if composite action between the joists and the floor occurred during the tests.

(2) Compare the stresses at failure with the ultimate values given in Table B.1.

Load-deflection curves for the three laboratory tests are given in Figures 37, 38, and 39. From these data, the average stiffness, k_a for each of the three test structures was computed using the following expression.

$$k_a = \frac{\Delta W}{y_c} \tag{C.1}$$

where ΔW = load increment, seven blocks were used per load increment in the laboratory tests.

y_c = average incremental midpoint deflection obtained by averaging incremental deflections for the given load test.

The resulting stiffnesses are as follows.

Test 1, joists 10 and 11 - k_a = 3509.92 lb/inch

Test 2, joists 12 and 13 - k_a = 3372.22 lb/inch

Test 3, joists 14 and 15 - k_a = 3236.33 lb/inch

For a simply-supported and uniformly loaded beam, the midpoint deflection y, is

$$y = \frac{5(\Delta W)\ell^3}{384\ E\ I} \tag{C.2}$$

where E = modulus of elasticity of Jack Pine, $E = 1.35(10)^6$ psi see Table B.1

I = moment of inertia of the beam cross section

ℓ = span length, ℓ is estimated at 9.65 ft = 115.80 inch

Substituting equation (C.1) into (C.2) and solving for $I = I_t$, i.e., test moment of inertia, we obtain

$$I_t = \frac{5\ell^3 k_a}{384\ E} = 0.01498k_a \qquad \text{(C.3)}$$

Table C.1 compares I_t with I_j and I_c, where I_j is the moment of inertia of two 1.625 inch x 5.625 inch joists. I_c is the (composite) moment of inertia based on two joist and 2-inch-thick flooring for test structures 1 and 2, and two joists and 1-inch-thick flooring for test structure 3.

$I_j = 48.20\ (\text{inch})^4$

I_c (test structures 1 and 2) = $260.52\ (\text{inch})^4$

I_c (test structure 3) = $197.46\ (\text{inch})^4$

TABLE C.1

COMPARISON OF MOMENTS OF INERTIA

Test	I_t (inch)^4	I_t/I_j	I_t/I_c
1	52.58	1.09	0.20
2	50.52	1.05	0.19
3	48.48	1.01	0.25

These results indicate that composite action was very small because the ratios I_t to I_j are much closer to unity than are the ratios of I_t to I_c. So, in fact the floor specimen behaved more like two joists than a composite floor section. The contribution of the flooring to the strength of the floor section and to the effective moment of inertia of the floor section was small.

Average load-deflection curves for the three test structures are shown in Figure C.1. They are compared with a theoretical load-deflection curve which was computed on the assumption that the flooring did not contribute to the moment of inertia of the cross section.

Table C.2 compares computed shear and flexural stresses, t and σ, with corresponding ultimate values, F_b and F_v given in Table B.1.

TABLE C.2

COMPARISON OF STRESSES

Test	Failure Load lb/ft	Shear Stress, τ psi	F_v psi	τ/F_v	Flexural Stress, σ psi	F_b psi	σ/F_b
1	679.64	282	715	0.39	5805	5150	1.13
2	611.05	255	715	0.36	5246	5150	1.02
3	392.82	163	715	0.23	4034	5150	0.78

Since the ratios of σ/F_b are close to unity, test structures 1 and 2 failed in flexure. The reason for the low σ/F_b ratio for test structure 3 is that this test was not carried to completion. It will be recalled (see Chapter 4) that this test was terminated after joist 14 failed. There was no sign of distress in joist 15.

In Table C.2 the failure load is the applied load and does not include the dead load of the test structure. Stresses given in this table include the effect dead load. The dead load was 32.5 lb/ft for tests 1 and 2 and 19 lb/ft for test 3.

C.2 First Load Test

The first in situ load test is described in section 3.5. The aim of this section is to predict the load-deflection behavior of the joists up to failure. Joist 12 is analyzed. The reason for choosing this joist is that it is located the furthest from obstructions such as edges and partitions (see Figures 4 and 19). Also, the load-deflection data at its girder support are more complete then in the case of joist 11 for example (see Figure 5).

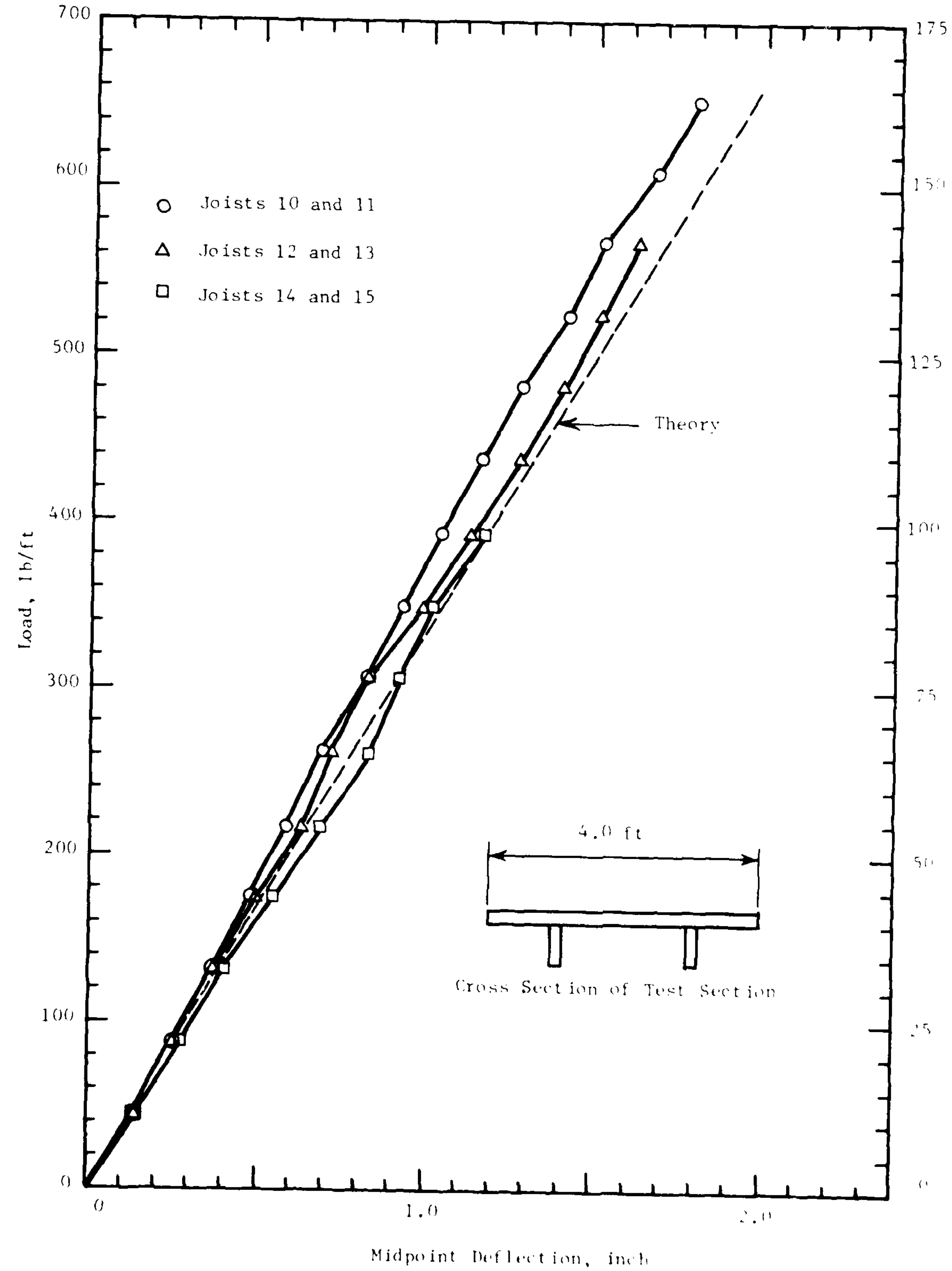


Figure C.1 Variation of Load versus Midpoint Deflection, Average Values

Load-deflection curves for joist 12 are given in Figure A.3(e). Load deflection curves for the girder in the vicinity of joist 12, i.e., at measuring positions 36 and 37 (see Figure 5) are given in Figure A.8(c) and Figure A.8(d) respectively. A deflection profile for joist 12 is shown in Figure C.2. Note that the girder deflection is fairly significant in comparison with the joist deflections.

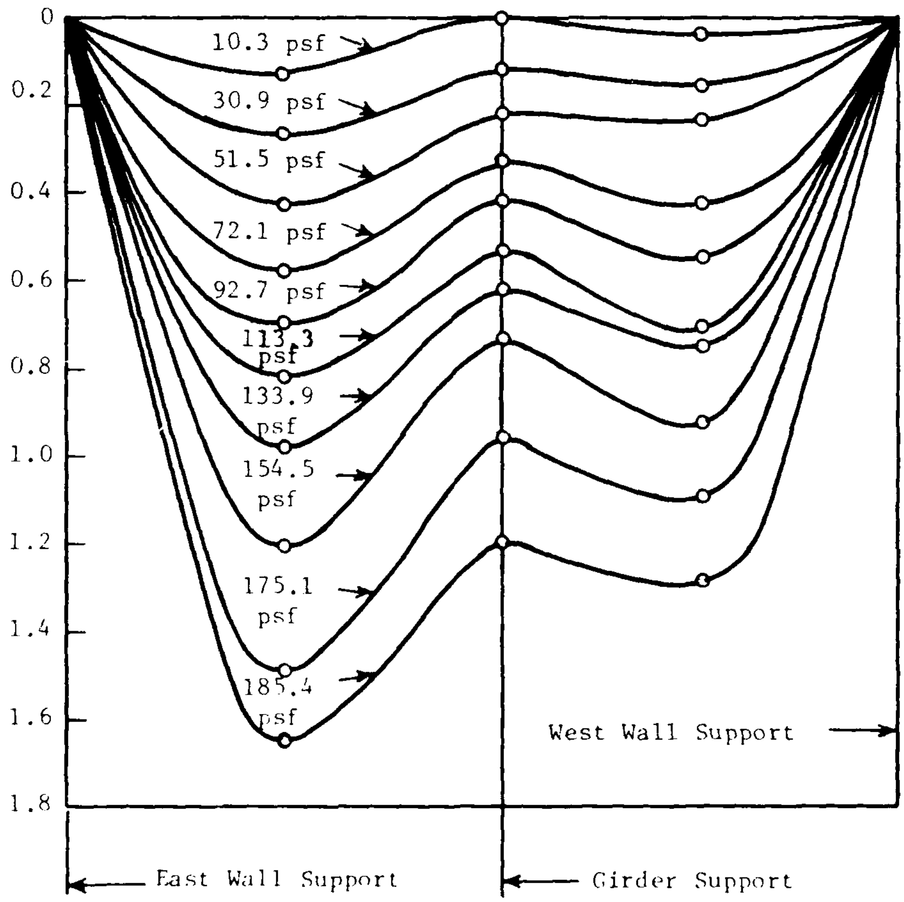


Figure C.2 Deflection Profile, Joist 12

The analytic model chosen for the analysis of the joist is shown in Figure C.3. The stiffness of the joist is simulated using a linear spring. The moment of inertia of the joist is constant and is based on the cross-sectional dimensions of the joist itself, i.e., the influence of the flooring is ignored. Equations for bending moments and deflections used in the analysis are given.

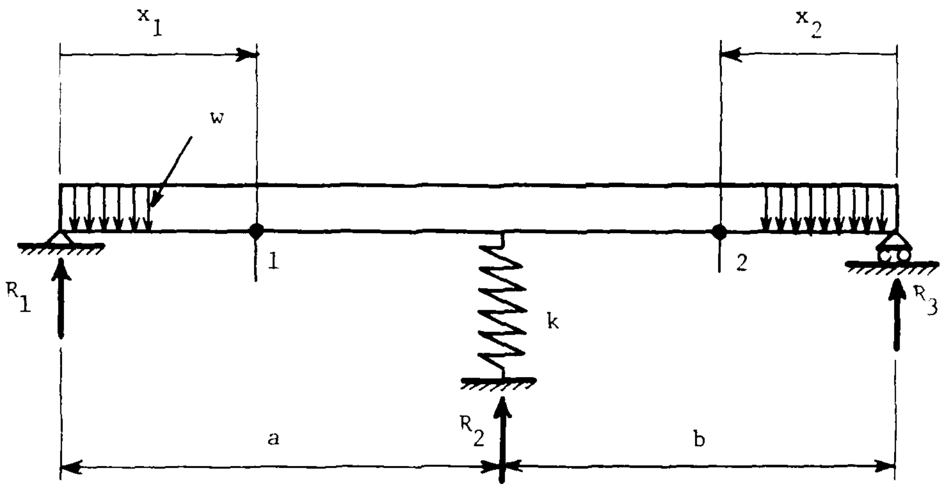


Figure C.3 Analytic Model of Joist

The maximum bending moment, M_{sp} in the span with length "a" is

$$M_{sp} = \frac{R_1^2}{2w} \tag{C.4}$$

where R_1 = left reaction (see Figure C.3)

w = unit load, lb/inch

$$R_1 = \frac{w\ell}{2} - \frac{R_2 b}{\ell} \tag{C.5}$$

where R_2 = girder reaction at the joist

ℓ = joist length, $\ell = a + b$

$$R_2 = \frac{w\ell}{8ab^2}(\ell^3 - 2\ell a^2 + a^3) - \frac{3EI\ell\Delta}{a^2b^2} \tag{C.6}$$

where Δ = girder deflection at the joist.

The bending moment at the girder support, M_{su} is

$$M_{su} = a\ b\ (\frac{w}{2} - \frac{R_2}{\ell}) \qquad (C.7)$$

Deflection at x_i (i = 1,2) is

$$y_i = \frac{1}{24EI} \left[w(\ell_s^3 x_i - 2\ell_s x_i^3 + x_i^4) - 4M\ell_s x_i (1 - \frac{x_i^2}{\ell_s^2}) \right] \qquad (C.8)$$

where ℓ_s = span length, i.e., $\ell_s = a$ for i = 1,

$\ell_s = b$ for i = 2

x_i = position where deflection is computed (i = 1,2)

In the analysis the following data were used:

a = 120.48 inch

b = 108.72 inch

ℓ = 229.20 inch

x_1 = 60.50 inch

x_2 = 54.00 inch

I = 24.10 $(\text{inch})^4$

E = 1.35 $(10)^6$ psi

Δ = Experimental value of girder deflection at position 37, Figure A.8(d).

The reason for using experimental values of girder deflections in the analysis rather than computing them as part of the analysis is that girder deflections could not be accurately computed. Recall that the girder was made up of five separate pieces of wood nailed together. During the loading process some separation of these pieces occurred as shown in Figure C.4. Part of the joist deflection at the girder is attributable to the flattening of the girder cross section due to the separation of the constituent pieces.

Figure C.4 Separation of Girder During First Load Test (note cracked column plate)

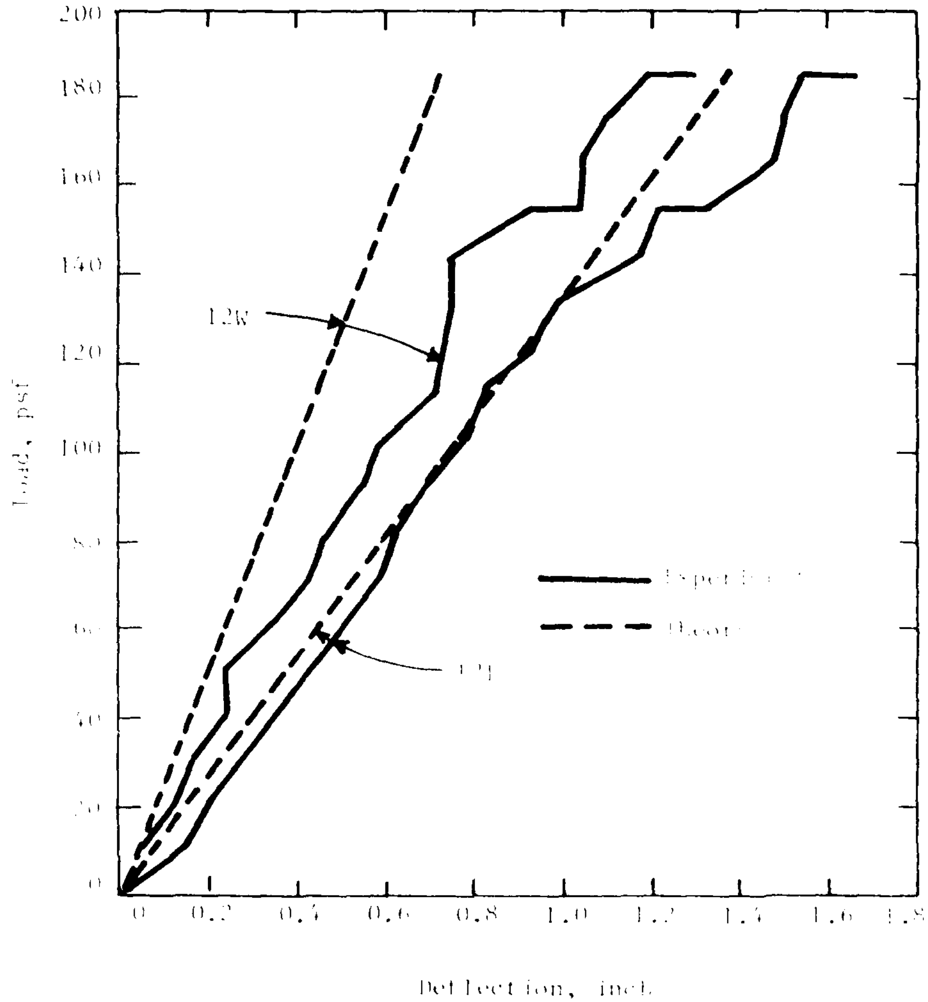


Figure C.5 Midpoint Deflections of Joist 12

Figure C.5 compares experimental results and analytic predictions for the deflections of joist 12. In the east span the comparison is good, while in the west span the results are fairly far apart. The reason for this is not clear, however it is possible that the load was not uniform as was assumed in the analysis.

The average time duration for the first load test was approximately 30 hr (1.25 days). For this duration, F_b = 4600 psi, see Table B.1. The maximum bending stress in span a, σ_{sp}, and the bending stress at the support, σ_{su} have the following values for a uniform load of 185.4 psf.

$$\sigma_{sp} = 4226 \text{ psi}, \quad \sigma_{sp}/F_b = 0.92$$

$$\sigma_{su} = 5140 \text{ psi}, \quad \sigma_{su}/F_b = 1.12$$

Thus at the girder the joist was 12 percent over the predicted rupture stress F_b, while in span "a" it was 8 percent below F_b.

Joist 12 did not fail, while similarly loaded joists 13 through 17 failed at 185.4 psf. These joists failed within the long span (span a) rather than at the girder where the stress was higher. The reason for this may be attributed to the fair number of natural and man-made defects in each of these joists except joist 14. The reason that joist 12 did not fail may be because joists 13 through 17 failed first resulting in a load shift and a reduction in the load on joist 12. Also, joist 12 had very few defects. Figure C.6 is a view looking toward the north wall and showing joists 12 through 17.

C.3 Second Load Test

The second load test is described in section 3.8. The aim of this section is to estimate the ultimate, uniform load capacity of the expediently upgraded floor system. An analysis of the columns, girder, upgrading studwall verticals and floor joists indicates that the floor joists are the weakest elements in the floor system. The capacity of the joists was determined using the model and results shown in Figure C.7. All joists are assumed to be identical and the load is assumed to be uniform. Flexibility of supports is neglected. With the latter assumption, the estimated load carrying capacity is a lower bound for static load. The load is assumed to have a duration of 1.25 days. From Figure C.7 the maximum bending moment, M_{max} and maximum shear V_{max} have the values

$$M_{max} = 394.40\ w$$

$$V_{max} = 36.67\ w$$

where w is the uniform load per unit length.

Figure C.6 View Looking Toward North Wall (Joist 12 in foreground)

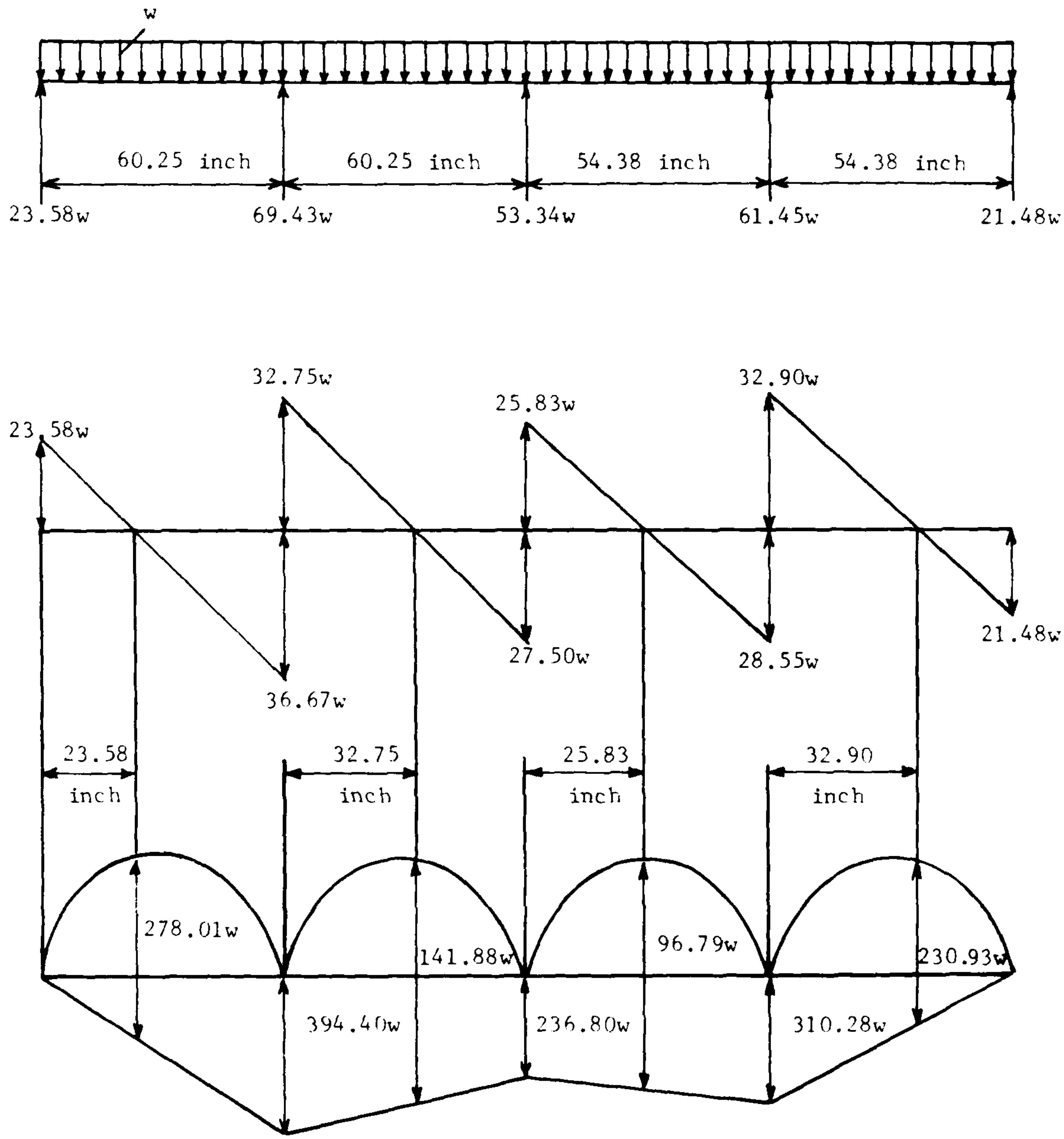


Figure C.7 Joist Loading, Shear and Bending Moment Diagrams

From Table B.1 the ultimate stresses for flexure and shear for the 1.25 days load duration are the following.

$$F_b = 4{,}600 \text{ psi}$$

$$F_v = 480 \text{ psi}$$

With these values the corresponding net uniform load required to produce failure of the floor system in flexure and shear is estimated as

$$q_f = 4.11 \text{ psi} = 591.50 \text{ psf (flexure)}$$

$$q_v = 3.27 \text{ psi} = 470.40 \text{ psf (shear)}$$

The load sustained without failure in the second load test was 559.3 psf which is larger than the predicted failure load. Some reasons for this discrepancy could be:

1. The predicted failure load is a lower bound because the deformation of the joist supports is neglected.
2. It is difficult to achieve a truly uniform load using concrete block when the blocks are not sufficiently separated and the height exceeds some two or three layers. Interaction between the blocks when the floor deforms can result in a redistribution (arching) of the load over the floor system.

APPENDIX D

PROBABILITY OF PEOPLE SURVIVAL

This appendix contains estimates of the probability of people survival if this basement is used as a personnel shelter against blast effects in the as-built or the upgraded condition. The overpressure required to produce collapse of the floor system over the basement when the load is dynamic, is estimated on the basis of ultimate stress for a 1 sec duration of load (see Table B.1). This approach for estimating the ultimate load is suggested in Ref 2. In estimating the probability of people survival, six assumptions are made:

1. The attack is produced by a single, 1-MT weapon detonated near the ground surface.
2. The upper story will be blown away without damaging the basement.
3. Windows to the basement are blocked.
4. Soil mounding to a height of 1 ft is provided all around the basement to protect the protruding basement wall.
5. People survival is dependent on the strength of the floor over the basement.
6. The floor joist is the weakest component among joists, girder and columns.

Probability of people survival, P(S) is estimated using the following procedure.

$$P(S) = P(S|\overline{C})\ P(\overline{C}) + P(S|C)\ P(C) \qquad (D.1)$$

where $P(S|\overline{C})$ = the probability of survival given that the structure does not collapse

$P(\overline{C})$ = probability of structure survival

$P(S|C)$ = probability of survival given that the structure collapses

$P(C)$ = probability of structure collapse, $P(C) = 1 - P(\overline{C})$

No fatality level casualties are expected prior to the collapse of the floor system and therefore $P(S|\overline{C})$ is equal to one. The probability of survival given that the structure collapses, $P(S|C)$ is estimated as illustrated in Figure D.1. This figure is a transverse elevation cross section through the basement with the variation of the probability of people survival superimposed on it. It is assumed that debris from the collapse of the floor system will mostly affect basement areas which are approximately halfway between the wall and the columns in the two spans. In these areas, people would be impacted and trapped by the collapsed floor and thus the probability of survival is assumed to be zero. Close to the walls and columns the probability of survival is assumed to be one. Between these locations a linear variation is assumed as shown in Figure D.1. In this analysis the average probability of survival for the whole basement is used and therefore P(S|C) = 0.5.

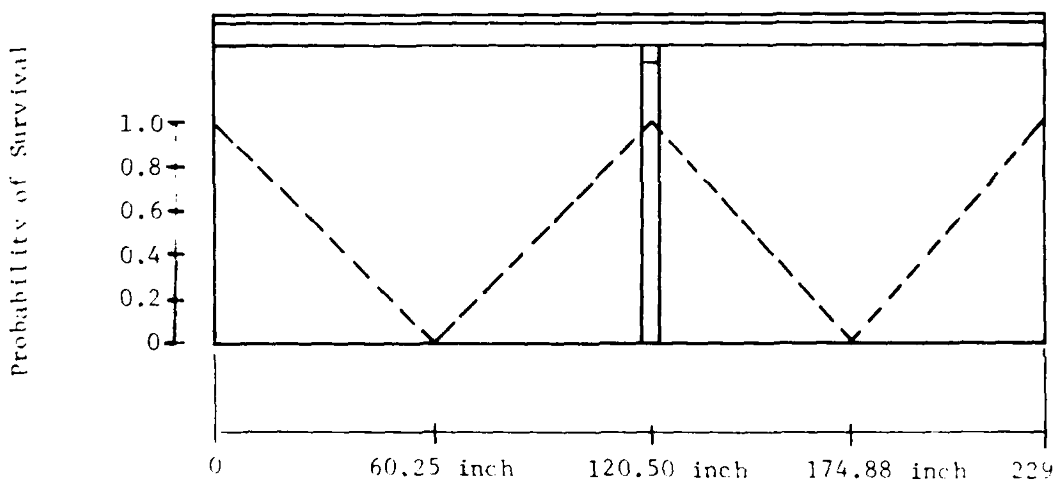


Figure D.1 Variation of the Probability of People Survival

Probability of structure survival, $P(\overline{C})$ is computed as

$$P(\overline{C}) = \Phi\left(\frac{\ln \tilde{\theta}}{\zeta_\theta}\right) \qquad (D.2)$$

where $\tilde{\theta} = \tilde{r}/\tilde{s}$, is the median safety factor

$\zeta_\theta = \sqrt{\ln[(1 + \Omega_r^2)(1 + \Omega_s^2)]}$, is the total degree of dispersion of the safety factor $\theta = r/s$

r = ultimate resistance of the structure

s = load on the structure (load and resistance are expressed in the same units and are statistically independent)

$\tilde{r}$ = median value of r

$\tilde{s}$ = median value of s

$\tilde{\theta}$ = median value of θ

Ω_r = coefficient of variation of the resistance

Ω_s = coefficient of variation of the load

The lognormal assumption of r and s is made in line with suggestions given in Ref 9.

Values of the ultimate resistance of the floor system are given in Table D.1.

TABLE D.1

ULTIMATE RESISTANCE OF THE FLOOR SYSTEM

	As Built	Upgraded
No Soil Load	1.5 psi	5.1 psi
With Soil Load	0.8 psi	4.4 psi

Based on these values and equations D.1 and D.2, probabilities of structure survival and people survival are given in Figures D.2 and D.3 respectively.

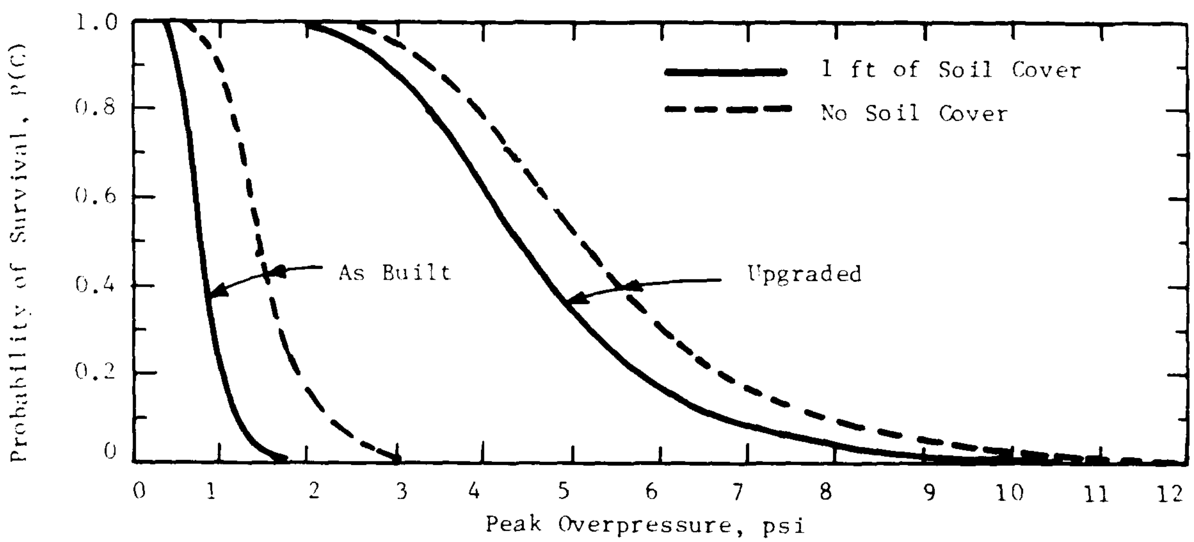


Figure D.2 Probability of Shelter Survival

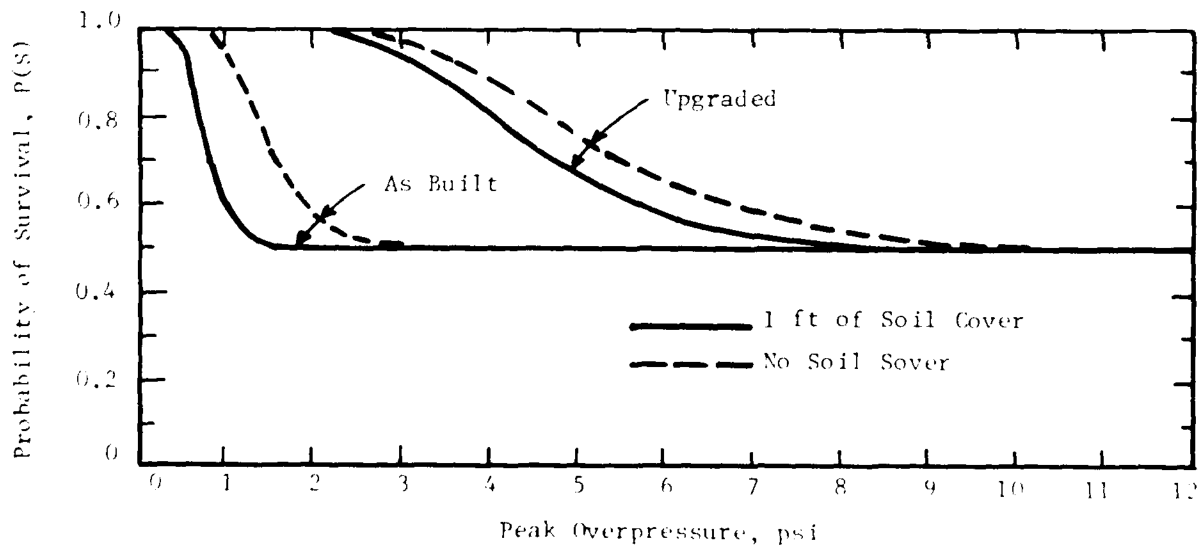


Figure D.3 Probability of People Survival

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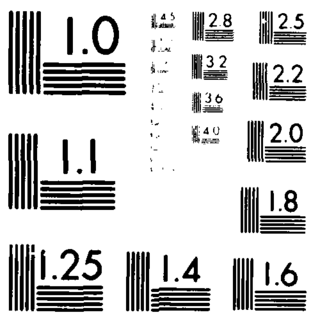

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LOAD TESTS OF A WOOD FLOOR OVER A BASEMENT (unclassified)

Final Report 87 pages

Contract DCPA01-78-C-0223 IIT Research Institute
June 1980

ABSTRACT: The results of an experimental study which was conducted to determine the strength of residential, wood floor systems over basements to uniform static load are presented in this report. A single family residence which was slated for demolition was acquired and load tested. Two load tests were conducted. The first load test was concerned with the strength of the as-built floor system. One half of the floor system was instrumented and loaded to collapse. Failure was experienced at 185.4 psf. The second load test was concerned with the strength of an expediently upgraded floor system. The remaining half of the floor system was upgraded by placing a studwall in the longitudinal direction halfway between the exterior wall and the girder in each of the two spans. The floor was loaded to 559.3 psf. At this load the test was terminated due to reasons of safety. The floor system did not fail. Additional tests were conducted in the laboratory on the unfailed portions of the floor system. This consisted of three "simple beam" tests of sample consisting of two joists with flooring attached. The load was uniformly distributed. The loading in each case was accomplished using solid concrete block.

This report includes experimental results, analysis of experimental results and predicted collapse loads using a simplified prediction method.

Probability of people survival estimates are included for two shelter conditions. In the first, the shelter is assumed to consist of the as-built basement with one foot of soil over the floor for radiation protection. In the second, the shelter is assumed to consist of the expediently upgraded basement with one foot of soil for radiation protection. The probability of people survival is estimated against blast effects produced by the detonation of a single 1-MT nuclear weapon.

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